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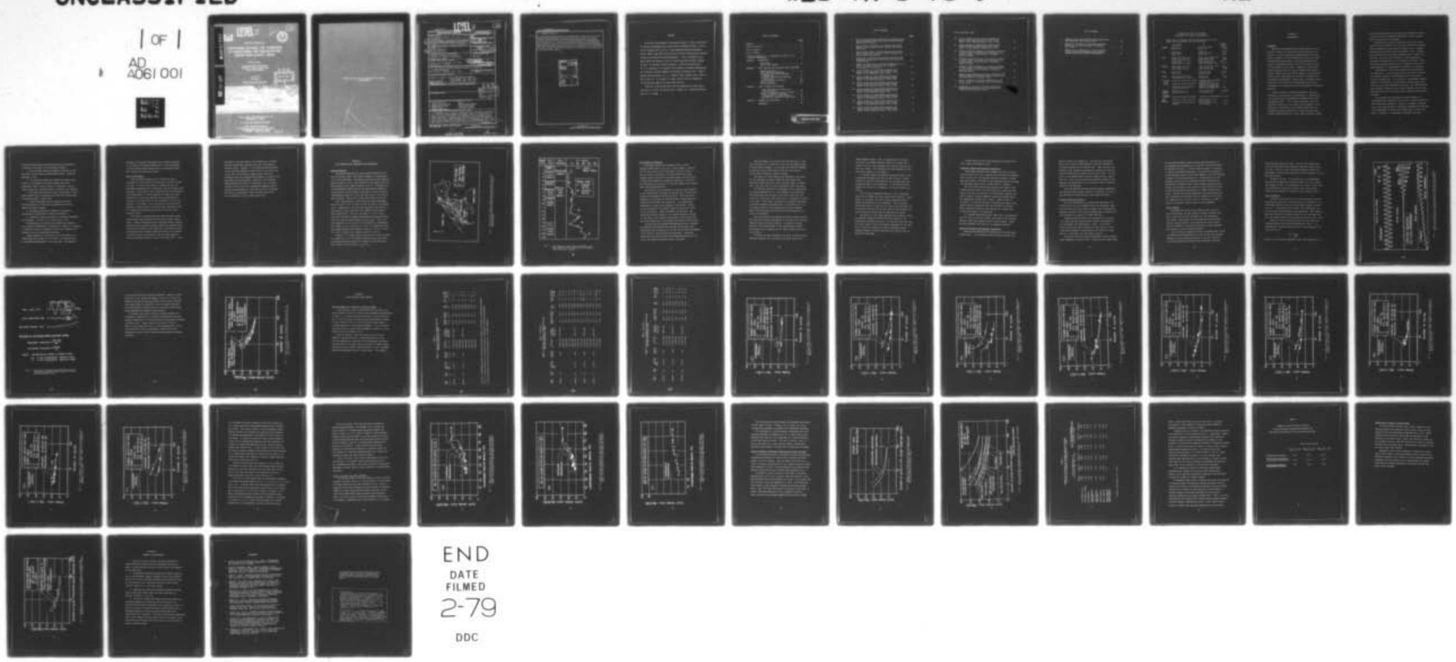

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TECHNICAL REPORT S-78-9

COMPARISON BETWEEN THE STRENGTHS OF UNDISTURBED AND RECONSTITUTED SANDS FROM NIIGATA, JAPAN

by

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August 1978

Final Report

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Undisturbed samples

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Laboratory cyclic triaxial strength tests were performed on undisturbed and reconstituted sand specimens from Niigata, Japan, to help evaluate how tests on reconstituted specimens can evaluate the cyclic strength of insitu cohesionless soils. Undisturbed specimens obtained from careful sampling with a large diameter sampler appeared to be of high quality, yet cyclic triaxial strengths measured in the laboratory were not particularly high. Specimens failed at cyclic stress ratios of about 0.15 at 20 stress cycles. CONT

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20. ABSTRACT (Continued).

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Reconstituted specimens prepared by pluviating sand through water were weaker than undisturbed specimens by factors of about 1.22 to 1.16. The cyclic strength difference between reconstituted specimens prepared by pluviating sand through water and reconstituted specimens prepared by moist tamping was about the same as the cyclic strength difference between reconstituted specimens prepared by pluviating sand through water and undisturbed field specimens. Thus sand reconstitution techniques such as wet tamping may better model insitu soil behavior than reconstitution techniques such as pluviation for sands such as these at Niigata, Japan.

PREFACE

This report was prepared by Professor Marshall L. Silver, University of Illinois at Chicago Circle, under Contract DACW39-76-M-2407, as part of the on-going work at the U. S. Army Engineer Waterways Experiment Station (WES), under CWIS 31145 Work Unit entitled "Liquefaction Potential of Dams and Foundations during Earthquakes." This investigation was made possible by support of the U.S.-Japan Cooperative Science Program of the U. S. National Science Foundation. The work was directed by Dr. W. F. Marcuson, III, Research Civil Engineer, Earthquake Engineering and Vibrations Division (EE&VD), Geotechnical Laboratory (GL). General guidance was provided by Dr. P. F. Hadala, Chief, EE&VD, and Mr. James P. Sale, Chief, GL. Mr. Ralph R. W. Beene, Office, Chief of Engineers, is the technical monitor for this CWIS work unit.

Directors of WES during this study and preparation of this report were COL G. H. Hilt, CE, and COL John L. Cannon, CE. Technical Director was Mr. F. R. Brown.

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CONVERSION FACTORS, US CUSTOMARY
AND METRIC TO SI UNITS OF MEASUREMENT

Metric and U.S. customary units of measurement used in this report can be converted to SI units as follows.

	TO CONVERT	TO	MULTIPLY BY
Length	inches (in.)	millimeters (mm)	25.40
	inches (in.)	meters (m)	0.0254
	feet (ft)	meters (m)	0.305
	miles (miles)	kilometers (km)	1.61
	yards (yd)	meters (m)	0.91
Area	square inches (sq. in.)	square centimeters (cm^2)	6.45
	square feet (sq. ft.)	square meters (m^2)	0.093
	square yards (sq. yd.)	square meters (m^2)	0.836
	acres (acre)	square meters (m^2)	4047
	square miles (sq miles)	square kilometers (km^2)	2.59
Volume	cubic inches (cu in.)	cubic centimeters (cm^3)	16.4
	cubic feet (cu ft.)	cubic meters (m^3)	0.028
	cubic yards (cu yd.)	cubic meters (m^3)	0.765
Mass	pounds (lb)	kilograms (kg)	0.453
	tons (ton)	kilograms (kg)	907.2
Force	one pound force (lbf)	newtons (N)	4.45
	one kilogram force (kgf)	newtons (N)	9.81
Pressure or Stress	pounds per square foot (psf)	newtons per square meter(N/m^2) or pascals (Pa)	47.9
	pounds per square inch (psi)	kilonewtons per square meter (kN/m^2) or kilopascals (kPa)	6.9
	kilogram force per square centimeter (kgf/cm^2)	Kilonewtons per square meter (kN/m^2) or Kilopascals (kPa)	98.07
Liquid Measure	gallon (gal)	cubic meters (m^3)	0.0038
	acre-feet (acre-ft)	cubic meters (m^3)	1233
Quantity of Flow	gallons per minute (gal/min)	cubic meters per minute(m^3/min)	0.0038
Unit Weight	pounds per cubic foot (pcf)	Kilonewtons per cubic meter (kN/m^3)	0.1572
	grams per cubic centimeter (gm/cm^3)	Kilonewtons per cubic meter (kN/m^3)	9.807

CHAPTER 1

INTRODUCTION

Background

Case histories describing earthquake effects on soil deposits clearly show that cyclic loading decreases soil strength. The most damaging strength changes have taken place in deposits of loose cohesionless soils where cyclic loadings have induced excess pore water pressures high enough to decrease effective stress to low values so that the soil behaves like a fluid. Examples of catastrophic damage resulting from this type of behavior were noted in Niigata, Japan during the 1964 earthquake, in San Fernando during the 1972 earthquake, and in other historical and contemporary earthquakes. Clearly then, an improved understanding of the stability of cohesionless soils during earthquakes is an important goal of geotechnical engineering research.

Several anlytical and design procedures have been developed to help predict the performance of cohesionless soil deposits during earthquakes as summarized by Valera and Donovan (1977). All of these procedures require an accurate evaluation of the insitu cyclic strength of soil materials. Potentially, insitu field testing that can evaluate the cyclic behavior of a large soil mass would provide the best measure of insitu cyclic strength. However,

no field testing procedure at present can adequately imput to a large soil mass the magnitude and form of seismic energy that is produced by an earthquake.

Alternatively, other more localized insitu test procedures are being considered for measuring insitu soil strength. These procedures include down-hole vibrator tests, standard penetration tests and the cone penetrometer tests. However, these procedures only evaluate the behavior of a small soil mass and often the test instrument or probe unnaturally influences the behavior of the surrounding soil mass. Thus, corrections between measured insitu strength values and the expected behavior of undisturbed soil elements are required.

Alternatively, cyclic soil behavior may be measured in the laboratory. By far the most widely used test to measure the dynamic strength of cohesionless soils is the cyclic triaxial strength test. In this test, a consolidated triaxial specimen is subjected to a periodically varying axial cyclic load wave form under undrained conditions. The resulting pore water pressure changes and cyclic axial deformations are then monitored with time to evaluate what is commonly called cyclic strength.

One approach for evaluating the insitu behavior of cohesionless soil elements in the field is to perform tests on undisturbed field specimens. However, to properly represent insitu soil characteristics, laboratory specimens must accurately reproduce both 1) insitu soil density and 2) insitu soil fabric. However, it is generally considered that good

undisturbed specimens of cohesionless soils are difficult to obtain in the field for the following reasons:

1. Even the best sampling procedures presently in use may densify loose sands and loosen dense sands. (Corps of Engineers, 1952).

2. The sampling process both reduces the insitu total stress on the specimen to zero and changes the insitu anisotropic state of total stress to an isotropic state of total stress with a possible change in measured soil behavior.

3. The sampling process may cause the specimen to loose some strength that has resulted from long term loading under a sustained stress. (Seed, 1976).

4. Sample disturbance during transportation and handling between the field and the laboratory can cause density and fabric changes.

5. Specimen extrusion and laboratory preparation techniques can significantly disturb the specimen.

6. Laboratory testing procedures can influence measured soil strength. (Silver, et al, 1976; Silver, 1977).

Consideration of the factors described above suggests that a better understanding of dynamic soil behavior is required to evaluate the cyclic strength of insitu soil elements from laboratory tests on undisturbed specimens.

Another approach for evaluating the insitu behavior of cohesionless soil elements in the field is to perform tests on reconstituted specimens in the laboratory. By preparing

specimens to controlled densities and by careful attention to the details of specimen preparation, it may be possible to form reconstituted specimens that can accuratelly model both insitu soil density and fabric.

Goal of this Report

From the above discussion it is clear that three techniques are being used to evaluate insitu cyclic strength of soils: 1) insitu field tests, 2) laboratory tests on undisturbed specimens and 3) laboratory tests on reconstituted specimens. Since each of these techniques has both advantages and disadvantages, it seems reasonable to expect that some combination of field and laboratory test procedures will continue to form the basis for the evaluation of cyclic soil strength. For example, field tests may be used to provide measures of insitu density and fabric while laboratory tests on reconstituted specimens may be used to provide cyclic strength values.

Such tests on reconstituted specimens can easily model insitu density but questions arrise on how to model insitu fabric. Fabric considerations are important because research has shown that different specimen preparation procedures and resulting differences in soil fabric can significantly influence measured soil strength values both under static loading conditions (Arthur and Phillips, 1975) and under dynamic loading conditions (Mulilis, et al, 1976; Ladd, 1974). Thus,

specimens at the same density but prepared with different specimen preparation techniques may well show different strength values. However, if the selected specimen preparation procedure adequately models insitu soil fabric, one of the major difficulties in applying the results of laboratory tests on reconstituted specimens to predict field performance can be minimized. Therefore, the following pages describe the results of laboratory cyclic triaxial strength tests on both good undisturbed specimens and on reconstituted specimens of loose sand to aid in evaluating how tests on reconstituted sands can help evaluate the cyclic strength of insitu cohesionless soils.

CHAPTER 2

FIELD SAMPLING AND LABORATORY TEST PROCEDURES

Sampling Location

Undisturbed specimens for testing were obtained from an area at Niigata, Japan, that showed evidence of liquefaction in the 1964 earthquake. The sampling site was located approximately 10 meters (33 ft) from the bank of the Shinano River on approximately level ground as shown in Fig. 1. Following the 1964 earthquakes, damage surveys reported severe evidence of liquefaction at this location in the form of surface cracking (approximately parallel to the river axis) and in the form of sand volcanoes. The area is in the flood plain of the river.

Up to approximately 5 years before the 1964 earthquake, this area was 4 m (13 ft) lower and was the bed of the Shinano River. Subsequently, this area was reclaimed, most likely by constructing a dike along the river channel and by dumping sand through water. Finally, the upper 1 m (3 ft) of the site was reclaimed by dumping miscellaneous borrowed materials after the site was raised above the river level. Thus, the soil deposition at the river site was probably 1) uncompacted fill dumped in air in the top 1 m (3 ft); 2) undensified fine sand dumped through water between a depth of 1 to 4 m (3 to 13 ft), and 3) fluvial river deposits below 4 m (13 ft).

Further, it is highly likely that the top of the river bed at a depth of 4 m (13 ft) might occasionally have been dried and desiccated during periods of drought in the recent past. Fig. 2 shows the soil profile at the site.

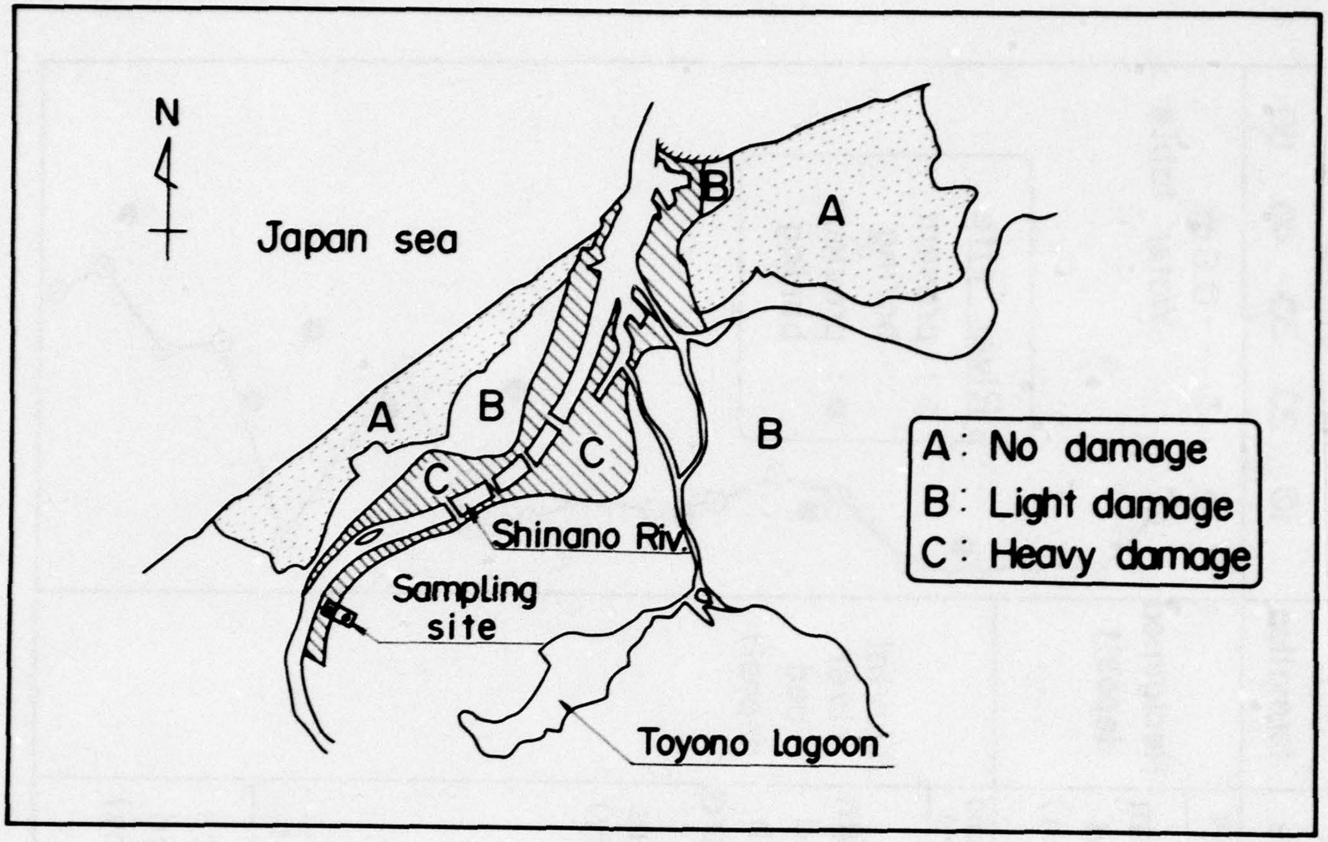


Fig. 1 Map of Niigata showing damage patters caused by the 1964 earthquake and the site where undisturbed sand samples were obtained.

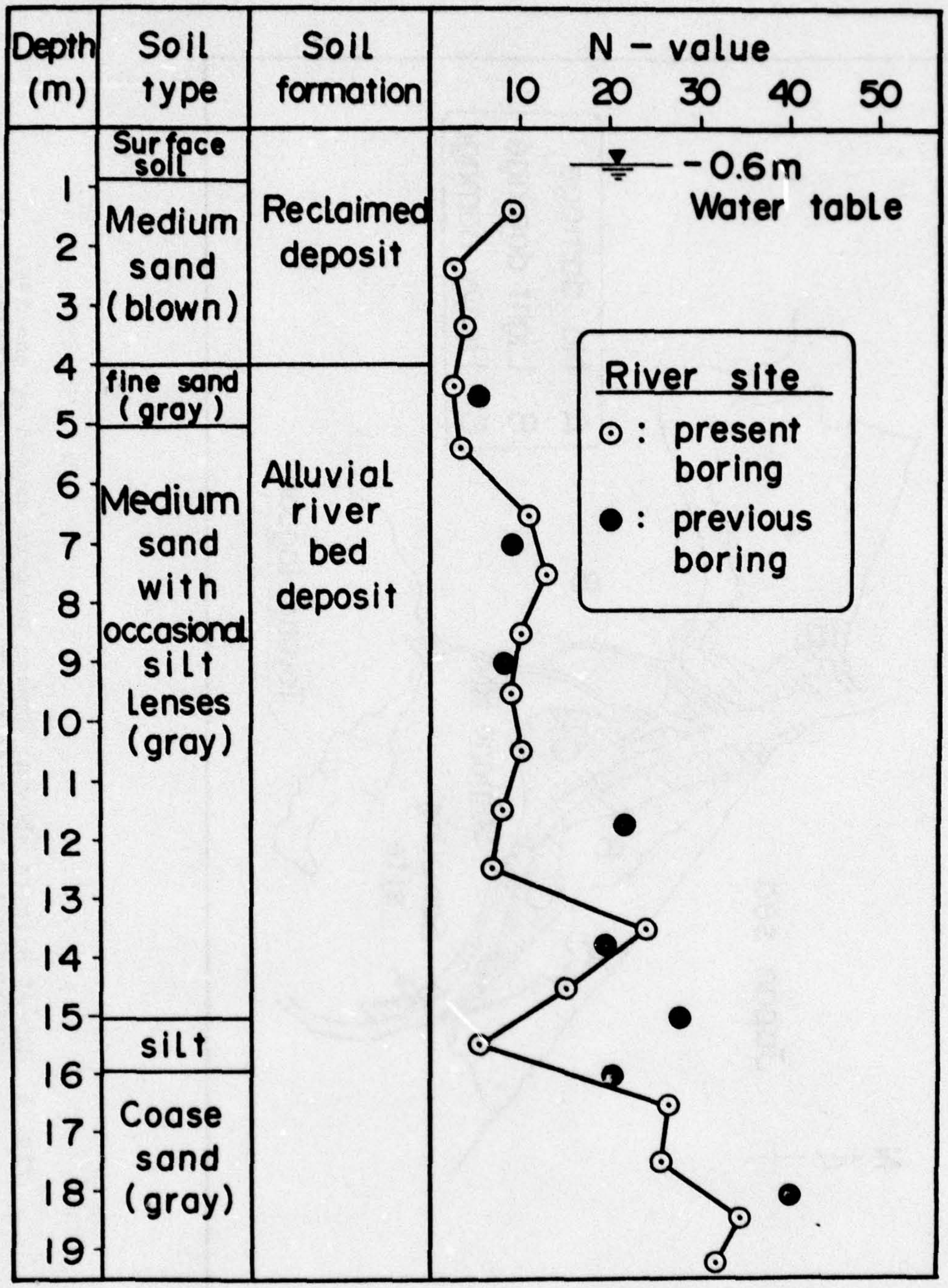


Fig. 2 Soil Profile at the River Site showing Soil Type, Method of Soil Deposition and Standard Penetration Test Values.

Field Sampling Procedures

Undisturbed samples were obtained using a large diameter sampler 200 mm (8.0 in) in inner diameter, 1000 mm (25 in) high, having a wall thickness of 8.2 mm (0.32 in). The core barrel consisted of two steel halves which were clamped together during the drilling process, but which were unlocked after sampling to expose the specimen in the field for evaluation of the quality of the soil sample and to obtain small undisturbed specimens.

The cutting bit at the bottom of the core tube contained a core catcher used to prevent the washing out of sand as the sampler was withdrawn from the bore hole. The core catcher consisted of two pieces of stainless steel screen which were folded and held within the cutting bit. The screens were connected by a cable that extended to the surface. During the coring process, the screens rested inside of the cavity in the cutting bit. After the sampler was advanced, the cables were pulled, closing in the screens securely, cutting off the sand at the bottom of the sample and restraining the sand from falling out as the sampler was lifted to the surface.

The bore hole was advanced conventionally with a fishtail bit modified with baffles that directed the drilling fluid upward away from the bottom of the hole so that sand disturbance at the bottom of the bore hole was minimized. Continuous casing was used and drilling mud was always maintained at the top of the casing to prevent caving of the sand.

When the sampler was removed from the bore hole, it was held vertically for at least ten hours to allow excess water to drain from the sample. In this way the capillary tensions of the pore fluid in the voids of the soil were used to prevent fabric changes or density changes.

The quality of these samples was considered to be excellent as indicated by visual inspection that showed that horizontal layers were kept intact and that lenses of coarse sand, fine sand, and silt were undisturbed, clear and sharp.

To avoid the problems of disturbance that are associated with the transportation of a large sample to the laboratory, small specimens were obtained in the field using thin 1 mm (0.039 in) thick brass tubes 50 mm (2 in) in diameter by 100 mm (4 in) long. These tubes, provided with a sharp cutting edge to minimize specimen disturbance and a longitudinal slit to aid sample extrusion, were pushed into the large diameter sample which was positioned horizontally and supported on a cradle for stability.

The core barrel of the sampler provided lateral restraint that prevented lateral displacement of the sand while the small diameter tubes were being inserted. In addition, outward movement of the back surface of the large diameter sample was prevented by a wooden plug held tightly against the rear sand surface.

To obtain the small specimens, the brass tubes were first inserted carefully into the sand at the front face of the

large diameter sample. Next, the upper half of the split core barrel was pushed backward exposing the intact sand surface, and the small specimens were carefully dug out from the large sample. This procedure was then repeated for the next intact portion of the sample.

To further protect the density and insitu fabric of the undisturbed specimens, they were quickly frozen in liquid nitrogen in the field and stored in dry ice while they were transported back to the laboratory. In the laboratory the samples were stored in a commercial ice cream freezer until tested.

This freezing technique turned out to be a reasonably simple way to handle the loose sands found at Niigata. Importantly, it was found that this freezing technique did not change the dimensions of the specimens. This is because drainage was used to clear the soil voids of excess water and only enough water was left at the grain to grain contacts to provide particle binding when the specimen was frozen.

The quality of the frozen specimens was checked by noting the specimen volume both before and after freezing. No measureable difference was noted. It was also found that the slit brass tubes of the large diameter specimens did not open up as a result of freezing and that the specimen length remained constant, further confirming that freezing did not disturb the specimens.

Further details of the field program are provided elsewhere (Silver and Ishihara, 1977).

Laboratory Undisturbed Specimen Preparation

Undistrubed specimens were prepared for cyclic triaxial strength testing by first trimming loose sand from the specimen ends to ensure that the specimen ends were parallel. Stones were then placed on the ends of the specimen which was extruded from the split brass tube by pushing down on a mandrel.

The still frozen undistrubed specimen was placed on the triaxial bottom platten, the top platten was lowered to make contact with the top stone and a split membrane explander was used to place a triaxial membrane around the specimen which was subsequently sealed with O-rings. A small vacuum of $-20\ KN/m^2$ (-5 in hg) was applied to maintain the shape of the sample which was allowed to thaw out from one to two hours before specimen dimensions were obtained to calculate the initial unit weight of the specimen.

All diameter measurements were taken with a circumference rule (Pi tape) at the top, middle, and bottom of the specimen. Specimen height was measured with a vernier caliper. Appropriate corrections were made for the membrane thickness.

Laboratory Reconstituted Specimen Preparation

Reconstituted specimens were prepared by pluviating sand through water. A pre-weighted amount of sand was mixed with

water to remove all trapped air. The sand was then poured slowly into a water filled membrane lined mold attached to the bottom platten of the triaxial cell. When required, the density of the specimen was adjusted by tapping the sides of the mold lightly with a hammer during mold filling. The top layer of the specimen was statically compacted with a small thin rod. Then, the top platten was carefully placed on the specimen and sealed with an O-ring before a small vacuum was applied to the specimen.

After the forming mold was removed, specimen dimensions were measured under a small vacuum of -20 KN/m^2 (-5 in hg) in the same manner as described for undisturbed specimens.

Triaxial Testing Procedures

For both undisturbed and reconstituted specimens, the triaxial cell was then assembled around the specimens, water was introduced into the triaxial chamber, and the vacuum was gradually reduced to zero while simultaneously increasing the cell pressure to a value of 20 KN/m^2 (400 psf).

Carbon dioxide was used to aid saturation by allowing it to flow from the bottom platten through the specimen to the top platten for approximately 1 hour. In this way, carbon dioxide was used to replace air from the soil voids. Since carbon dioxide is significantly more soluble in water than air, saturation time was greatly reduced.

Saturation and back pressure procedures closely followed those suggested by Silver (1976). Saturation was accomplished

by concurrently applying cell pressure and back pressure to the specimen while maintaining an effective confining pressure of 20 KN/m^2 (400 psf). B value checks were made at intervals to monitor the saturation process. All tests were conducted at back pressure values of 100 KN/m^2 (2000 psf) and the resulting B values in all cases exceeded 0.97. Consolidation was subsequently carried out by increasing the cell pressure while maintaining a constant value of back pressure and while monitoring axial deformation and specimen volume change with time so that the specimen consolidated unit weight could be determined. During both the back pressuring and consolidation process, small axial correction loads were applied to the piston to compensate for the unlift force on the piston in order to achieve an isotropic state of stress on the specimen.

Cyclic Testing

To perform the actual cyclic triaxial test, the cell piston was locked, the cell pressure and back pressure lines were closed, and the cell was moved to a servo-hydraulic test frame. The actuator was then connected and the proper seating load was applied. Pore pressure and cell pressure lines were opened, the piston was unlocked, and the specimen was allowed to rest for several minutes.

Cyclic triaxial strength tests were performed under stress-controlled undrained conditions by first closing the specimen drainage line and second by applying a 1 hz sine load wave form while monitoring changing load, deformation, and pore

water pressure values with time. The test was stopped when either the specimen exhibited double amplitude strain values of ±10% or when 200 stress cycles were exceeded without the development of significant excess pore water pressures or large specimen strains.

Following the test, the triaxial cell was disassembled and the entire sample was carefully washed into a pan, dryed, and weighed in order to calculate dry unit weight. Grain size, relative density and specific gravity tests were then performed on each specimen to determine index property values.

Test Calculation

A typical time history plot of load deformation and pore pressure with time for a cyclic strength test performed on an undisturbed specimen from the large diameter sample is shown in Fig. 3. It may be seen that there is no significant load fall off when large specimen strains developed. This constant amplitude load wave form was recorded for all tests and thus the test results meet the test limits proposed by Silver (1976).

Values of cyclic stress and strain for each cycle were calculated using the definitions presented in Fig. 4. For conveneience, values of cyclic vertical stress applied to the stress ratio, SR

$$SR = \frac{\sigma_{d\ell}}{2\sigma_o'}$$

where $\sigma_{d\ell}$ is the single amplitude cyclic axial stress and σ'_o

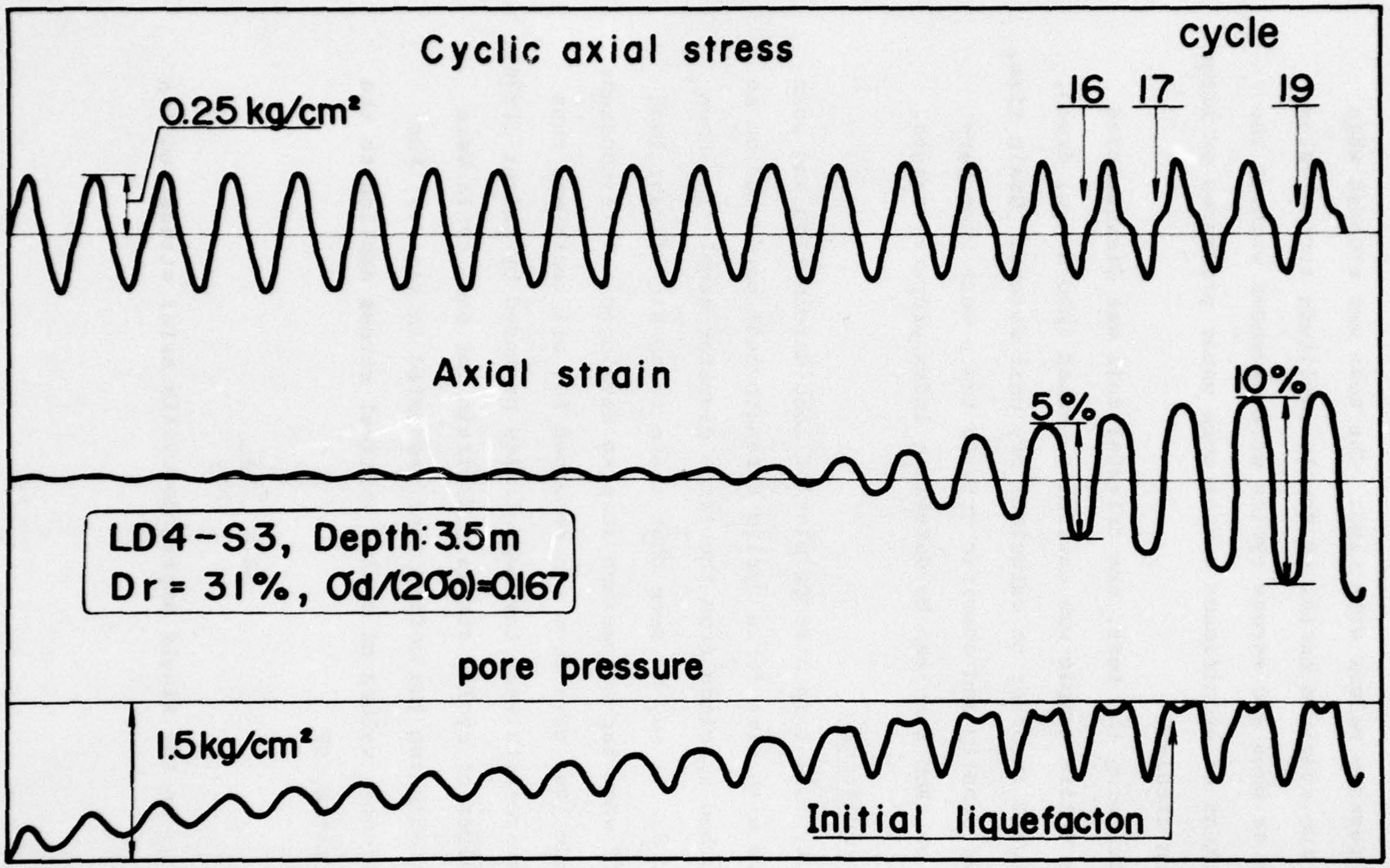
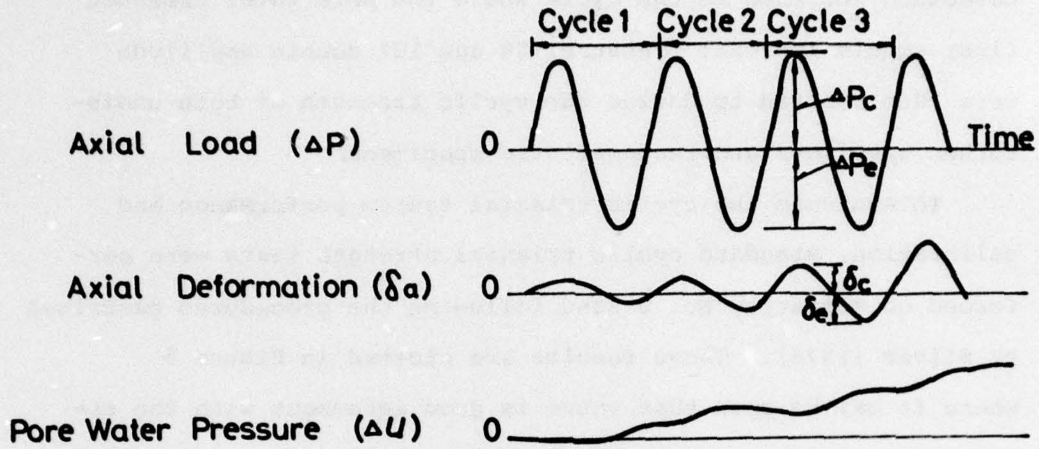


Fig. 3 Typical Time History of Load, Deformation and Pore Water Pressure for a Cyclic Triaxial Strength Test on an Undisturbed Specimen.



Definition of Calculated Stress and Strain Values

$$\sigma_{dl}\text{ (Single Amplitude)} = \frac{\Delta P_c + \Delta P_e}{2A_c}$$

$$\epsilon_a\text{ (Double Amplitude)} = \frac{\delta_c + \delta_e}{L_c}$$

Where $\Delta P_c, \Delta P_e, \delta_c, \delta_e$ are Defined in Figures Above

A_c is the Consolidated Specimen Area

L_c is the Consolidated Specimen Length

Fig. 4 Definition of Measured Load-Deformation Values and Calculated Stress Strain Values for Cyclic Triaxial Strength Tests

is the initial effective confining pressure. Values of cyclic stress ratio (SR) versus the number of cycles to initial liquefaction (defined as the cycle where the pore water pressure first equals the cell pressure) 5% and 10% double amplitude were then plotted to define the cyclic strength of both undisturbed specimens and reconstituted specimens.

To evaluate the cyclic triaxial system performance and calibration, standard cyclic triaxial strength tests were performed on Monterrey No. 0 sand following the procedures described by Silver (1976). These results are plotted in Figure 5 where it may be seen that there is good agreement with the results obtained by other laboratories, implying that test procedures used and system calibration meet generally accepted standards.

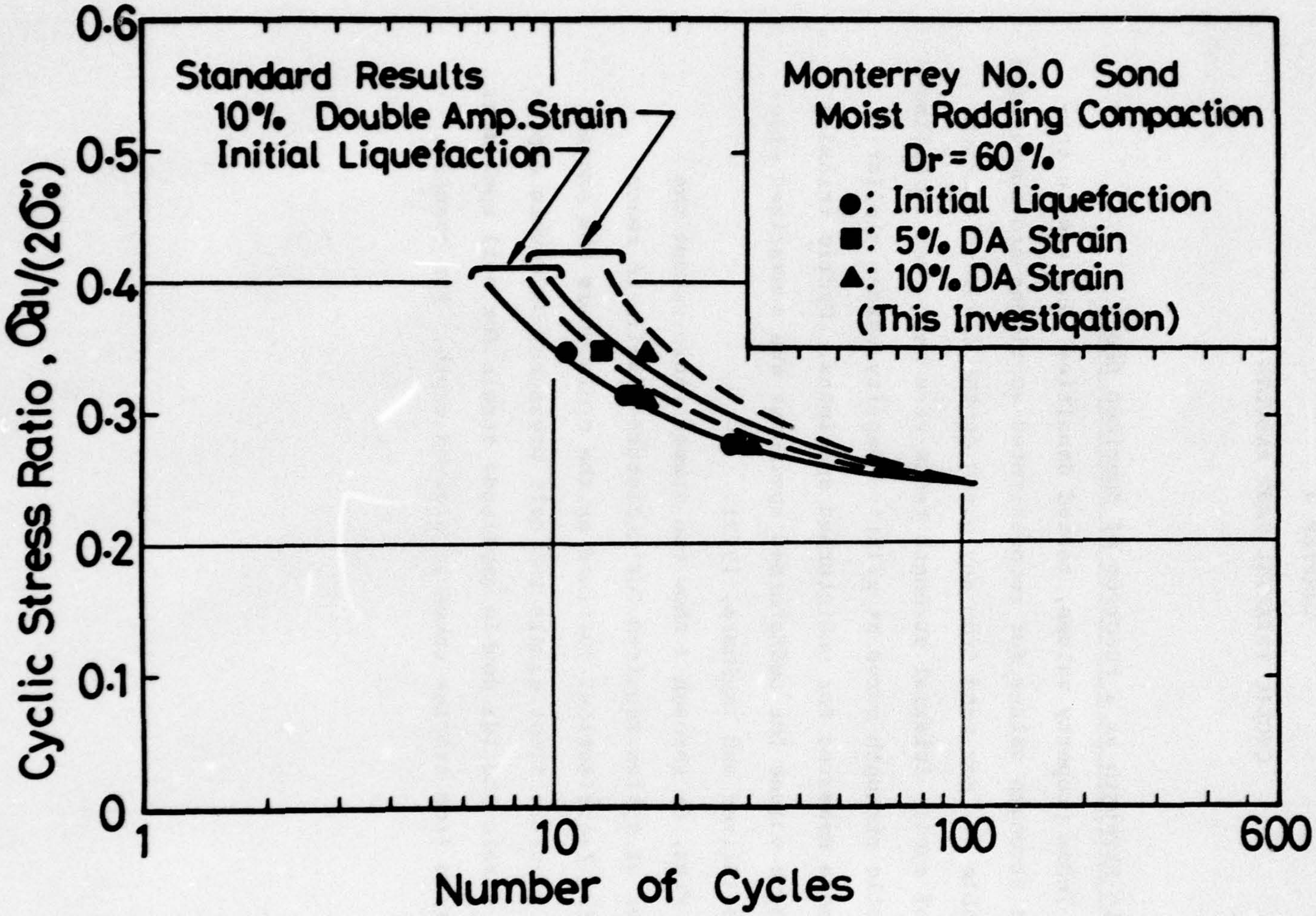


Fig. 5 Results of Check Tests Performed on Monterrey No. 20 Sand to Evaluate Cyclic Triaxial Equipment Performance and Calibration.

CHAPTER 3

CYCLIC TRIAXIAL TEST RESULTS

Cyclic Strength as a Function of Specimen Depth

Index property values, tested densities, and cyclic triaxial strength values for reconstituted specimens are summarized in Table 1. For sand from any given depth, a sufficient number of cyclic triaxial strength tests were performed to define a cyclic strength curve at relative density values similar to those measured for undisturbed specimens. Cyclic triaxial strength values for undisturbed specimens are summarized elsewhere (Silver and Ishihara, 1977).

Figs. 6a through i show the stress ratio versus the number of cycles required for undisturbed soils to reach initial liquefaction, (defined as the cycle where the excess pore pressure first equals the cell pressure) 5% double amplitude strain and 10% double amplitude strain for soil specimens obtained from samples taken at a given depth. For example,

TABLE 1

SUMMARY OF THE CYCLIC STRENGTH OF RECONSTITUTED SAND SPECIMENS FROM NIIGATA, JAPAN

Sample No.	Depth (m)	Gradation D50 (mm)	Gradation U_c	Specific Gravity	Void Ratio e_c	Void Ratio e	Limiting Void Ratios e_{max}	Limiting Void Ratios e_{min}	Relative Density D_{rc}(%)	Relative Density D_r (%)	$\frac{\sigma_{d\ell}}{\sigma_o'}$	No. of cycles init.	No. of cycles 5% DA	No. of cycles 10% DA
LD4-S2D	2.5	0.36	2.7	2.669	0.776	0.790	1.020	0.649	65.8	62.0	0.149	51	50	-
					0.855	0.873			44.5	39.6	0.168	4	3	-
					0.813	0.826			55.8	52.3	0.162	6	6	7
LD4-S3D	3.5	0.34	2.5	2.675	0.914	0.940	1.067	0.615	33.8	28.1	0.113	21	20	22
					0.899	0.925			37.2	31.4	0.106	68	68	-
					0.932	0.961			29.9	23.5	0.142	4	4	-
					0.922	0.951			32.1	25.7	0.093	107	107	-
					0.898	0.926			37.4	31.2	0.117	12	11	12

1) e_c; D_{rc} = Consolidated Void Ratio or Consolidated Relative Density of undisturbed or reconstituted specimens.

2) e, D_r = Initial Void Ratio or Initial Relative Density of undisturbed or reconstituted specimens.

3) All tests performed at an effective confining pressure σ_o' = 144 KN/m^3 (3000 psf).

TABLE 1 (Continued)

SUMMARY OF THE CYCLIC STRENGTH OF RECONSTITUTED SAND SPECIMENS FROM NIIGATA, JAPAN

Sample No.	Depth (m)	Gradation D50 (mm)	Gradation U_c	Specific Gravity	Void Ratio e_c	Void Ratio e	Limiting Void Ratios e_{max}	Limiting Void Ratios e_{min}	Relative Density D_{rc}(%)	Relative Density D_r (%)	$\frac{\sigma_{d\ell}}{\sigma'_o}$	No. of cycles init.	No. of cycles 5% DA	No. of cycles 10% DA
LD3-S2D	4.5			2.651	0.949	0.975	1.257	0.658	51.4	47.1	0.143	23	23	24
					0.974	0.991			47.2	44.4	0.165	14	14	16
					0.889	0.914			61.4	57.3	0.214	4	5	7
					1.022	1.053			39.2	34.1	0.146	11	11	12
					0.970	0.996			47.9	43.6	0.165	7	7	9
LD3-S3D	5.5	0.37	1.9	2.663	0.897	0.919	1.069	0.572	3.48	30.2	0.155	10	9	10
					0.801	0.821			34.0	31.5	0.138	5	5	7
					0.772	0.792			34.0	31.7	0.094	90	90	94
LD3-S4D	6.5	0.52	2.1	2.661	0.785	0.805	1.058	0.530	51.7	47.9	0.113	150	153	-
					0.795	0.816			49.8	45.8	0.184	3	3	4
					0.788	0.810			51.1	47.0	0.144	13	13	14
LD4-S7D	7.5	0.48	2.1	2.675	0.825	0.846	0.974	0.554	35.5	30.5	0.167	4	4	-
					0.773	0.791			47.9	43.6	0.140	134	134	136
					0.785	0.804			45.0	40.5	0.153	9	9	10

TABLE 1 (Continued)

SUMMARY OF THE CYCLIC STRENGTH OF RECONSTITUTED SAND SPECIMENS FROM NIIGATA, JAPAN

Sample No.	Depth (m)	Gradation D50 (mm)	Gradation U_c	Specific Gravity	Void Ratio e_c	Void Ratio e	Limiting Void Ratios e_{max}	Limiting Void Ratios e_{min}	Relative Density D_{rc}(%)	Relative Density D_r (%)	$\frac{\sigma_{d\frac{1}{2}}}{\sigma'_o}$	No. of cycles init.	No. of cycles 5% DA	No. of cycles 10% DA
LD2-S7D	8.5	0.50	2.7	2.672	0.833	0.859	0.988	0.552	35.6	29.6	0.157	5	6	6
					0.812	0.832			40.4	35.8	0.130	27	27	28
					0.807	0.830			41.5	36.2	0.118	37	38	38
LD2-S8D	9.5	0.75	3.1	2.650	0.770	0.796	0.997	0.477	43.7	38.7	0.163	6	6	7
					0.738	0.760			49.8	45.6	0.152	11	11	-
					0.757	0.781			46.2	41.5	0.141	39	40	41
LD2-S10D	11.5	0.48	3.0	2.678	0.765	0.784	1.022	0.544	53.8	49.8	0.157	8	10	12
					0.785	0.805			49.6	45.4	0.142	26	27	30

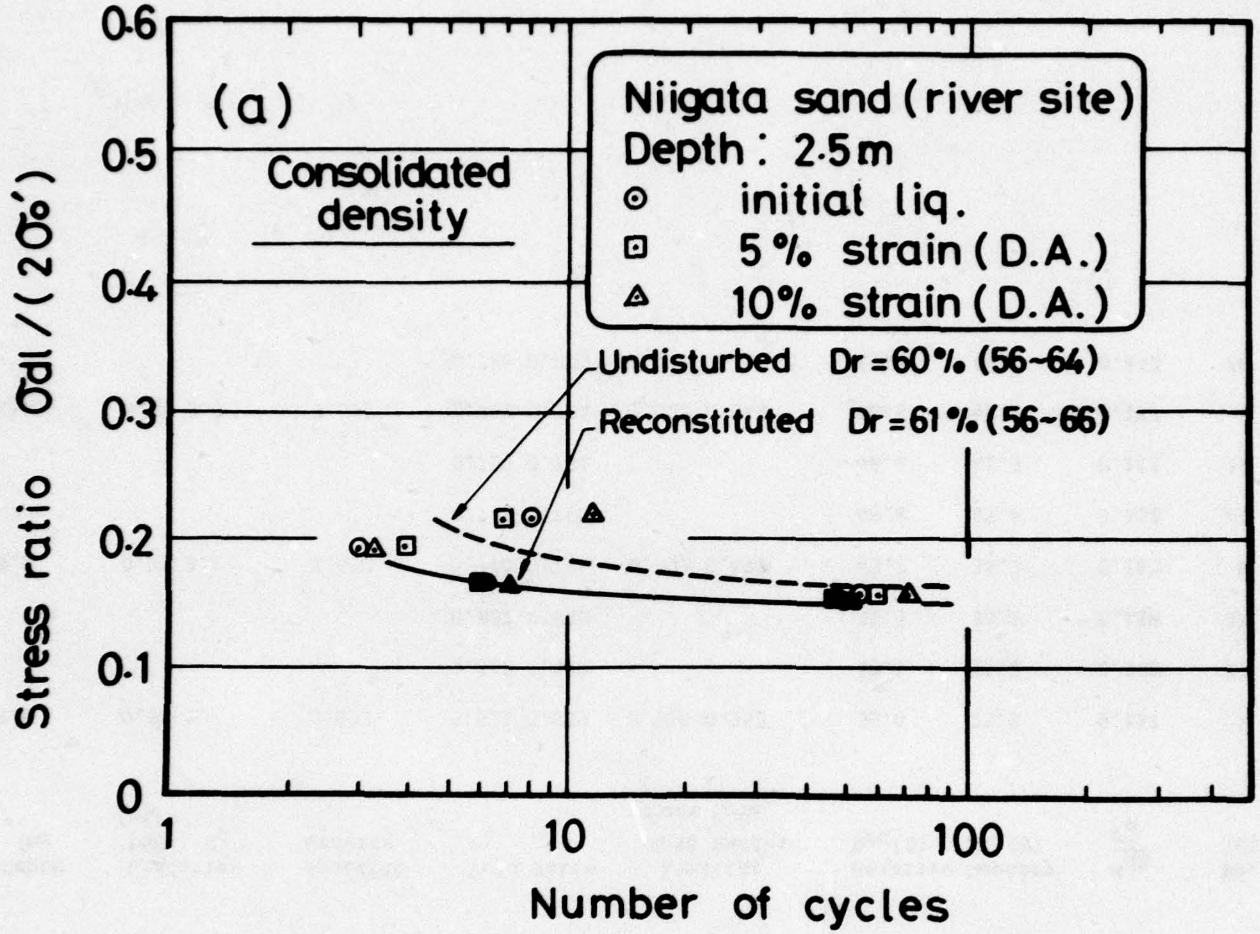


Fig. 6a Cyclic Strength of Undisturbed Specimens and Reconstituted Specimens, from Niigata Japan. Depth = 2.5 m, Relative Density = 60%.

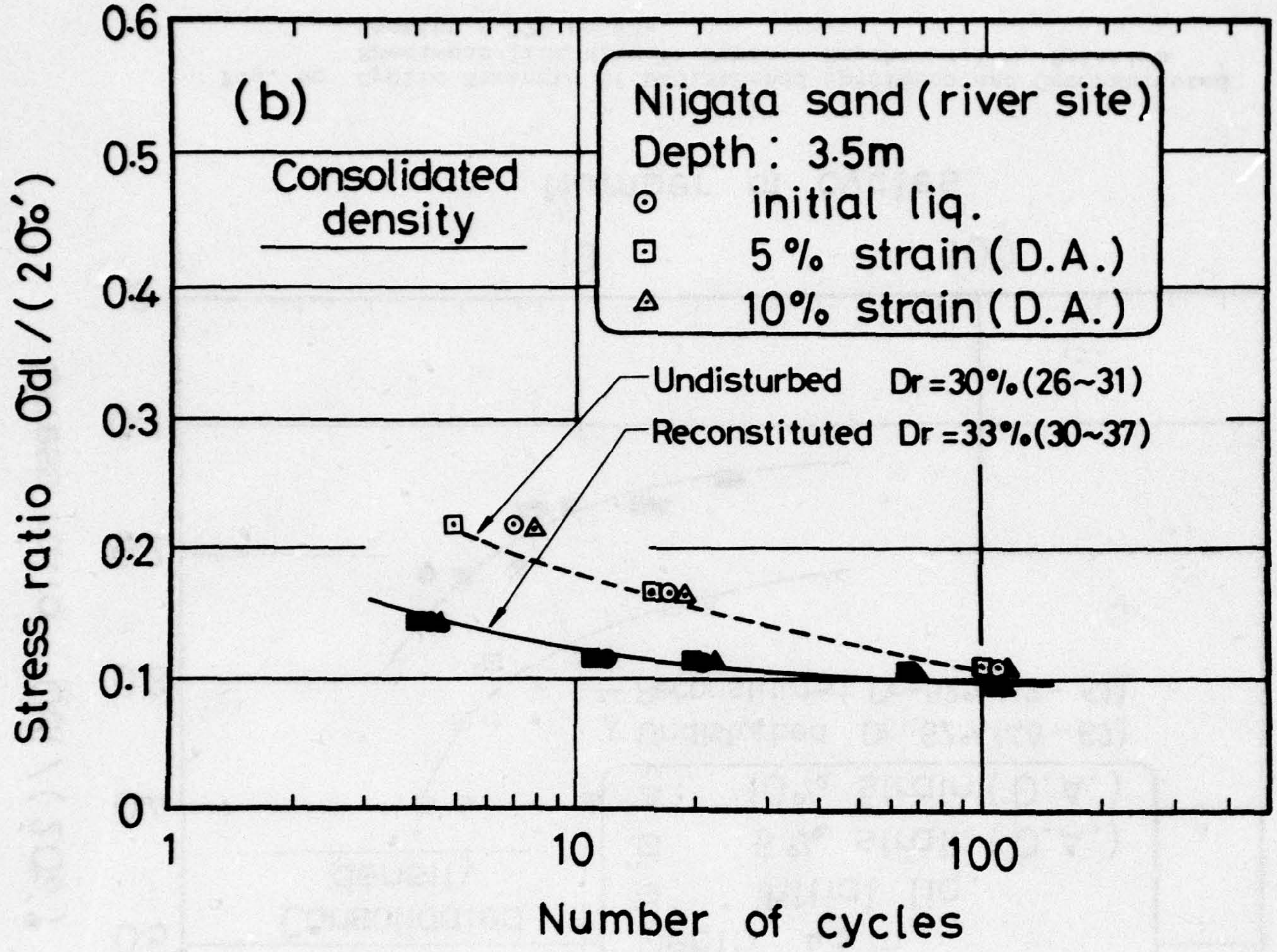


Fig. 6b Cyclic Strength of Undisturbed Specimens and Reconstituted Specimens from Niigata Japan. Depth = 3.5 m, Relative Density = 30% to 33%.

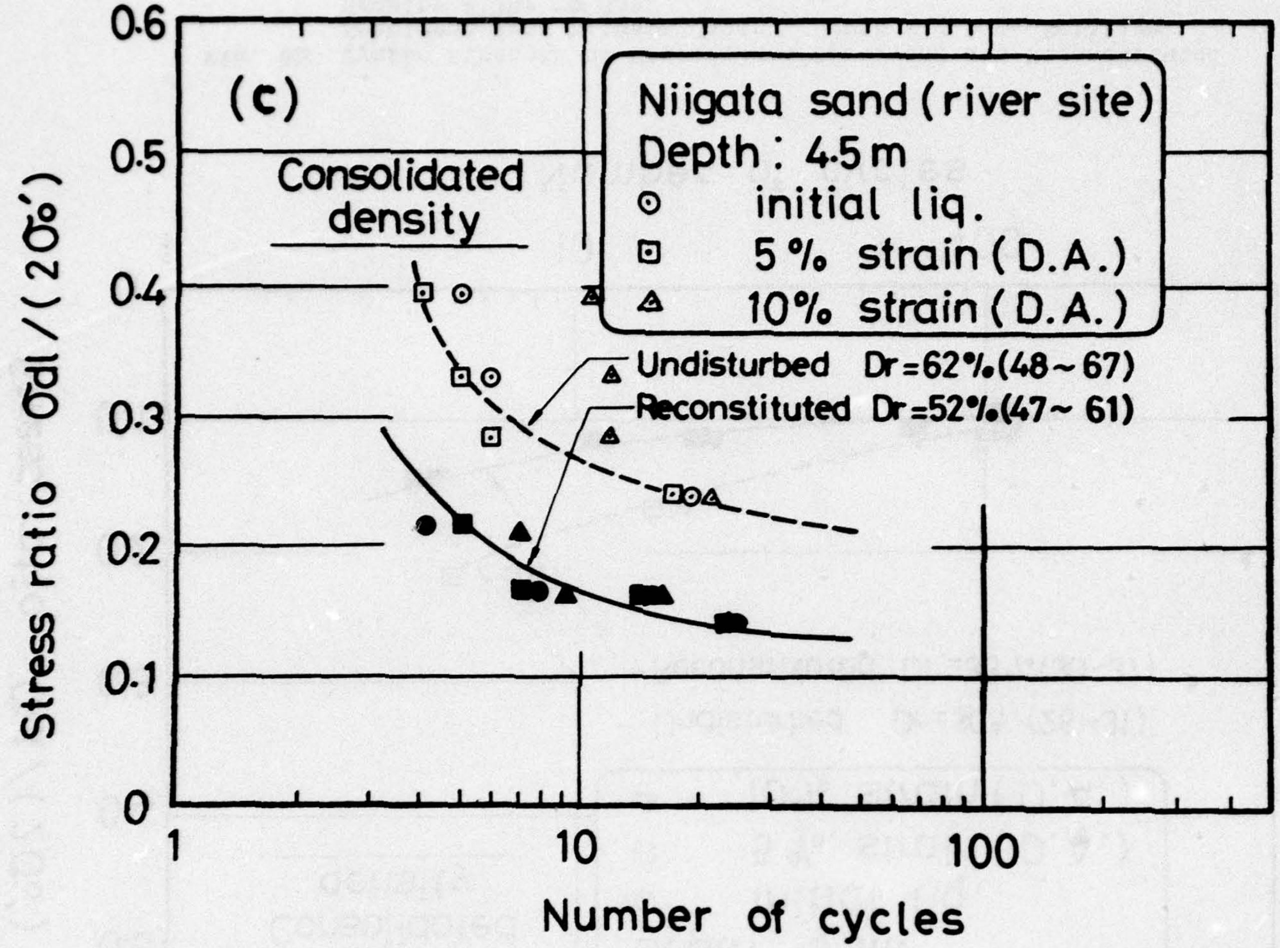


Fig. 6c Cyclic Strength of Undisturbed Specimens and Reconstituted Specimens from Niigata Japan. Depth = 4.5 m, Relative Density = 52% to 62%.

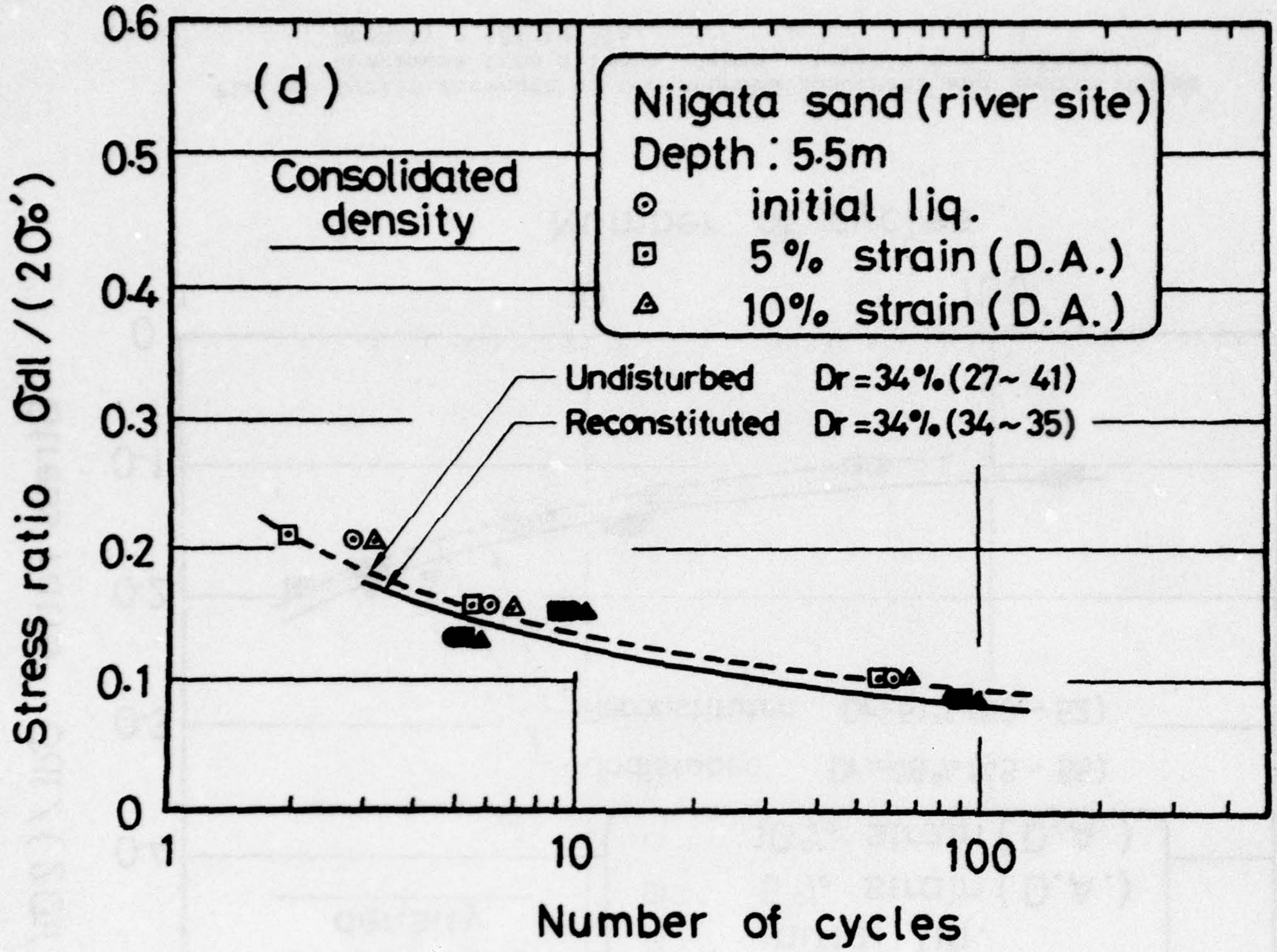


Fig. 6d Cyclic Strength of Undisturbed Specimens and Reconstituted Specimens from Niigata, Japan. Depth = 5.5 m, Relative Density = 34%.

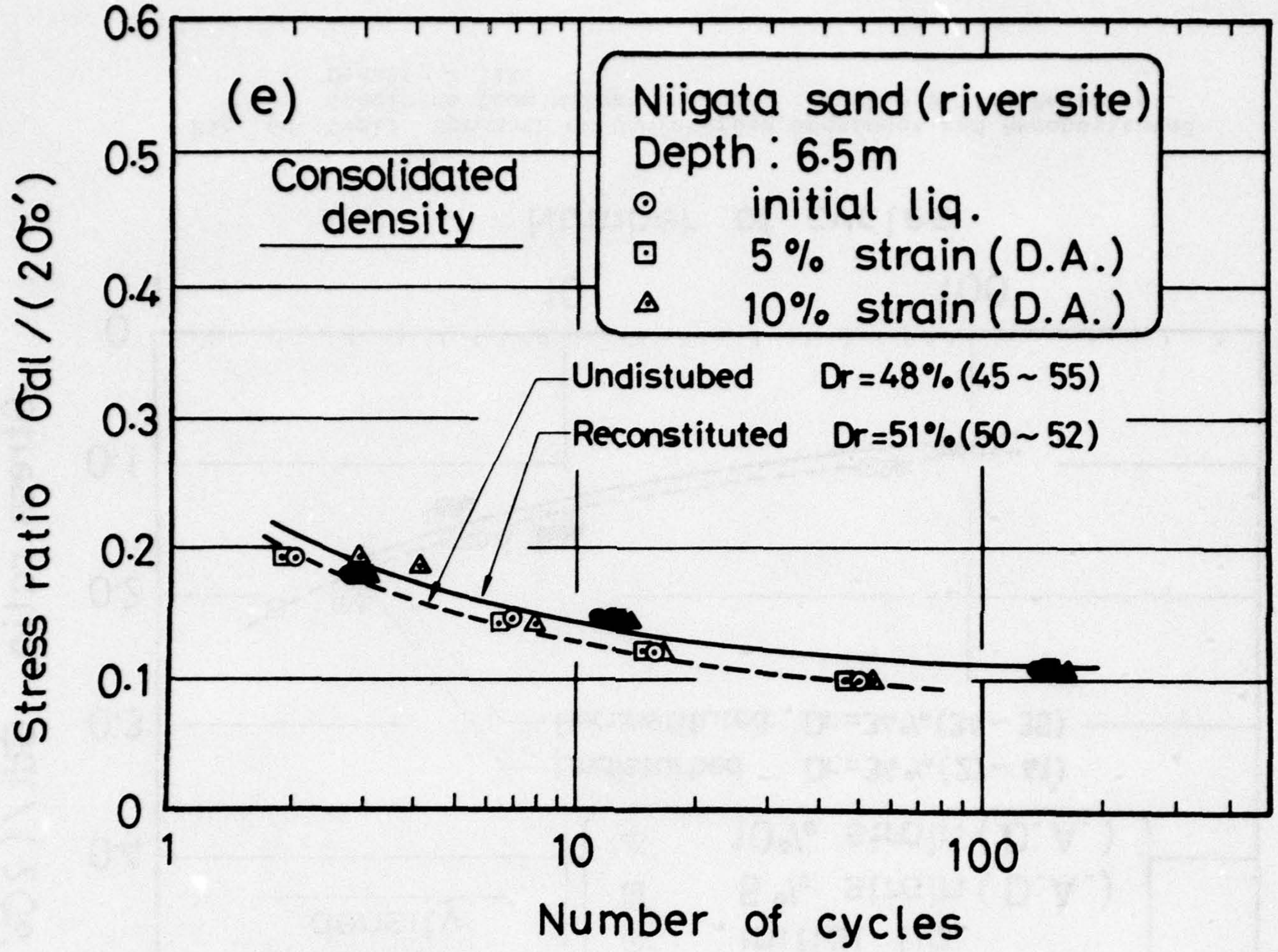


Fig. 6e Cyclic Strength of Undisturbed Specimens and Reconstituted Specimens from Niigata, Japan. Depth 6.5 m, Relative Density = 48% to 51%.

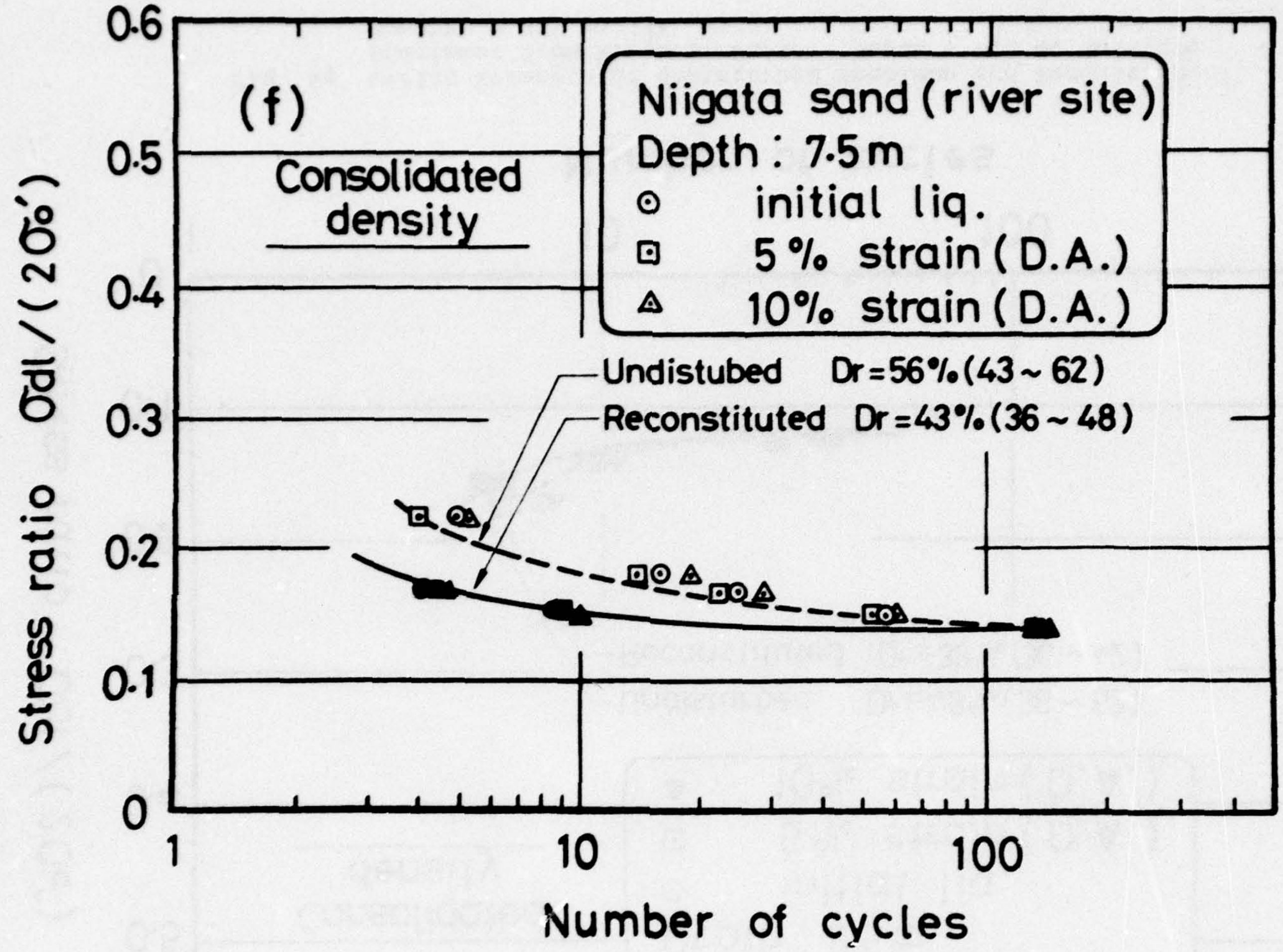


Fig. 6f Cyclic Strength of Undisturbed Specimens and Reconstituted Specimens from Niigata, Japan. Depth = 7.5 m, Relative Density = 43% to 56%.

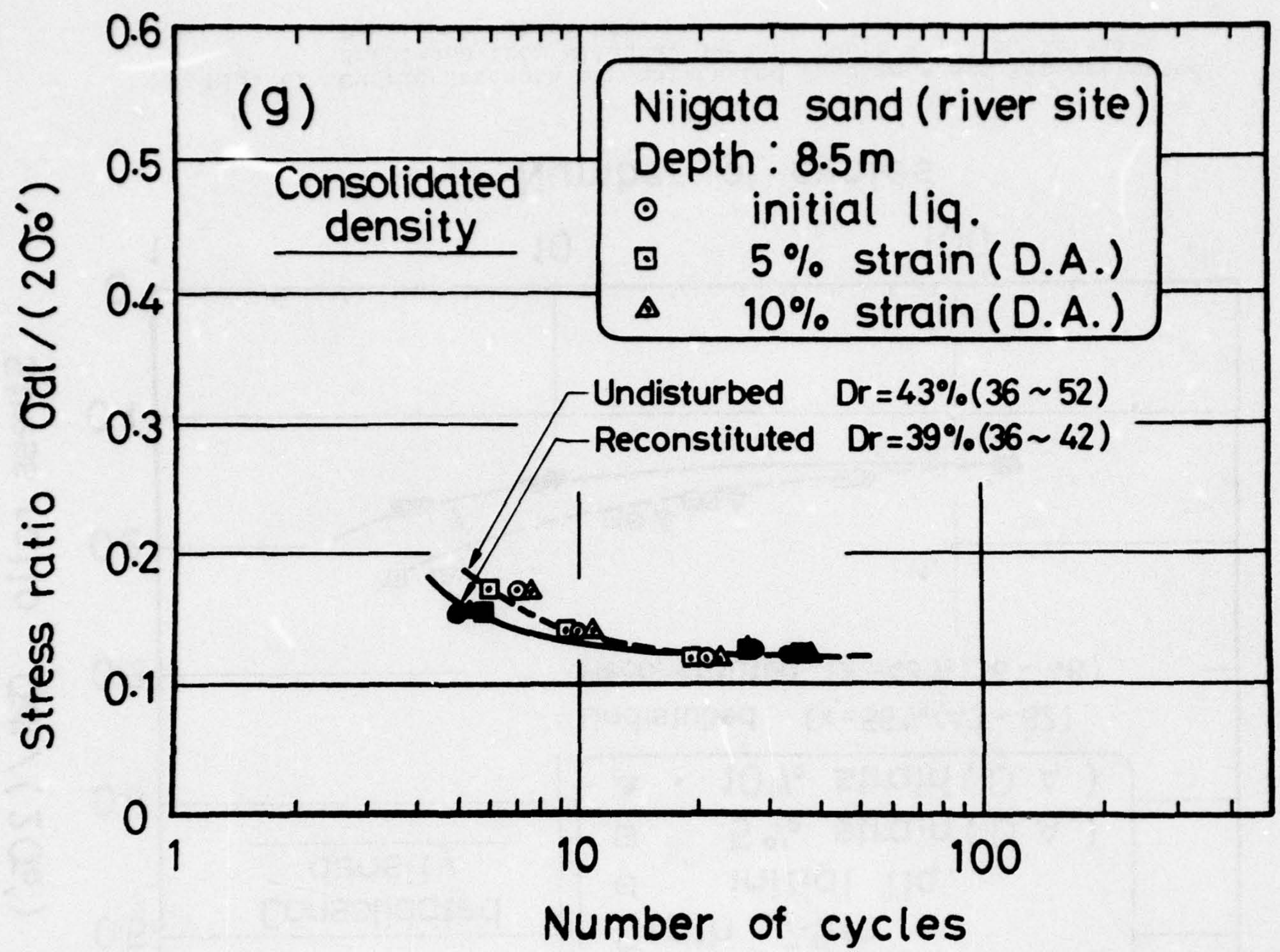


Fig. 6g Cyclic Strength of Undisturbed Specimen and Reconstituted Specimens from Niigata, Japan. Depth = 8.5 m, Relative Density = 39% to 43%.

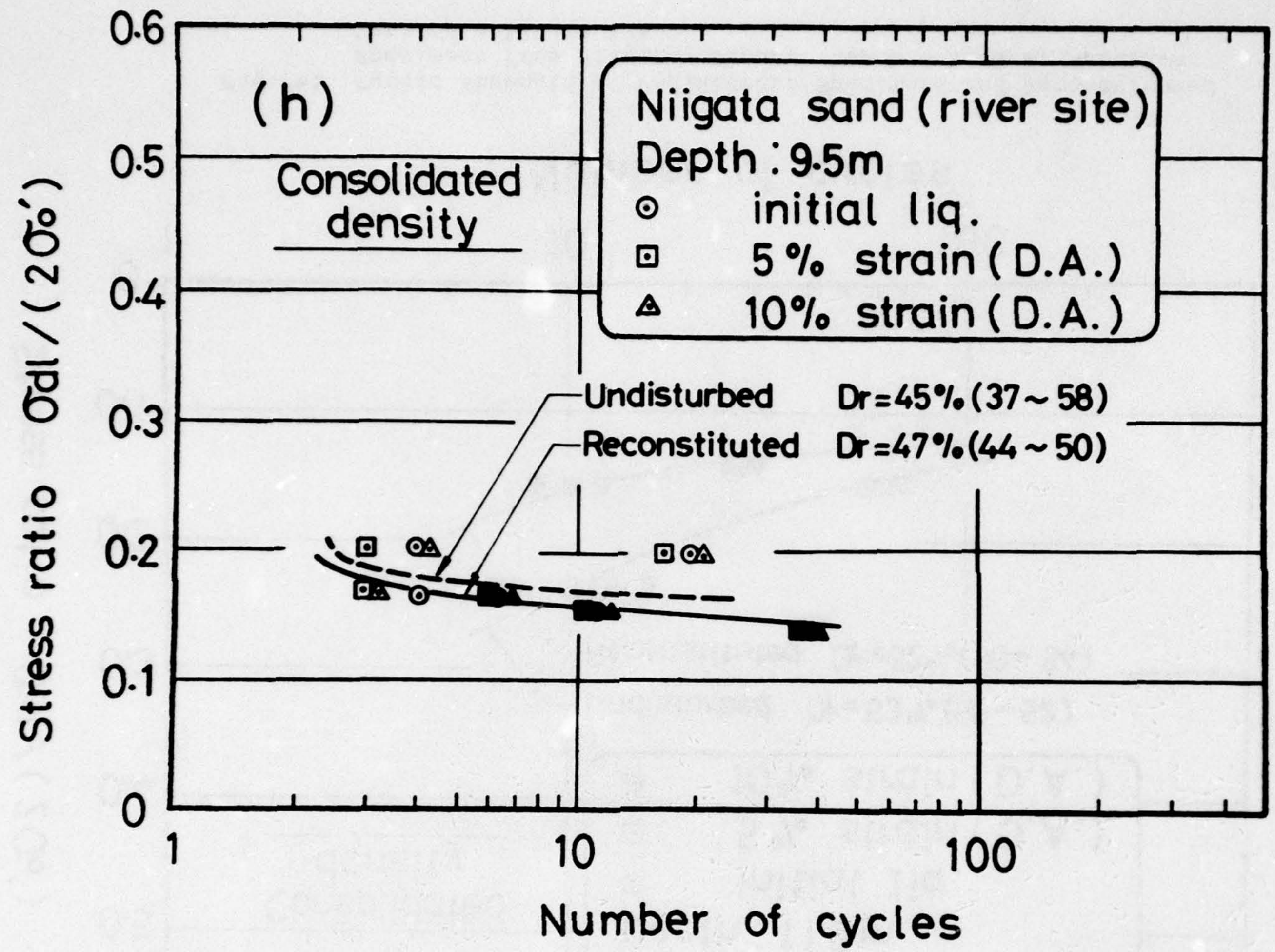


Fig. 6h Cyclic Strength of Undisturbed Specimens and Reconstituted Specimens from Niigata, Japan. Depth = 9.5 m, Relative Density = 45% to 47%.

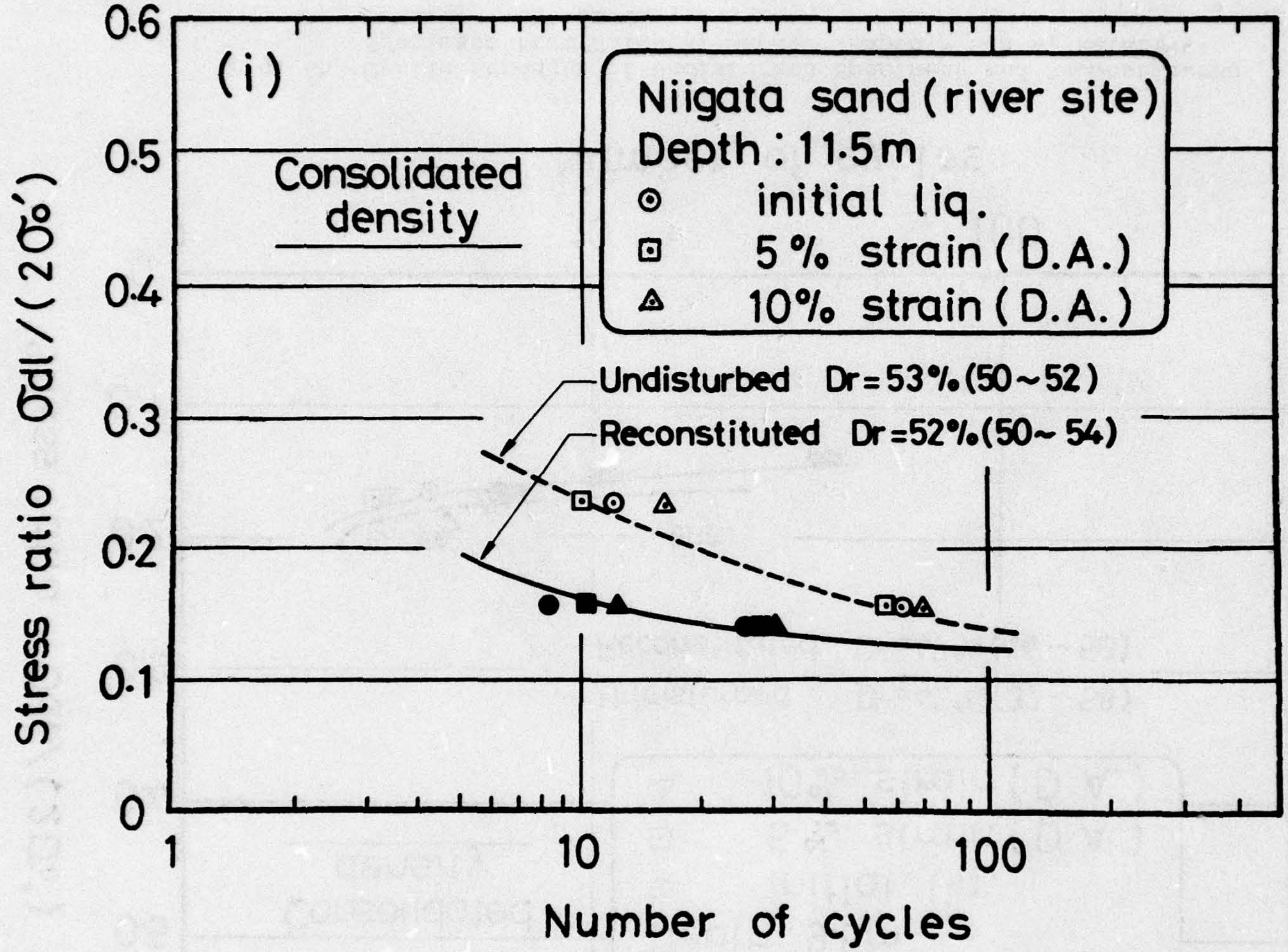


Fig. 6i Cyclic Strength of Undisturbed Specimens and Reconstituted Specimens from Niigata, Japan. Depth = 11.5 m, Relative Density = 52% to 53%.

Fig. 6a shows the cyclic strength of soils from a depth of 2.5 m where three small undisturbed specimens were obtained and tested. The relative density of these specimens ranged between 56% and 64%; therefore the average relative density for undisturbed specimens at this depth was 60% as shown on the Figure. It should be noted that these relative density values are for consolidated specimens and thus represent the strength of undisturbed specimens having a slightly higher density than insitu soils. It may be seen on the Figure that the cyclic triaxial strength curve plotted for failure defined as 5% double amplitude strain is rather flat and that at 20 cycles the stress ratio required to cause 5% double amplitude strain was on the order of 0.18.

Similar data for soils at a depth of 3.5 m is shown on Fig. 6b where it may be seen that the average tested relative density for the three specimens from this depth was 30% and that at 20 cycles the stress ratio required to cause 5% double amplitude strain was on the order of 0.16.

Figs. 6c to i show similar data for undisturbed specimens at successively deeper depths of 4.5 m, 5.5 m, 6.5 m, 8.5 m, 9.5 m, and 11.5 m. It may be seen from these figures that in general the cyclic stress ratio required to cause failure in 20 cycles was on the order of 0.14 to 0.18, except at a depth of 4.5 m where significantly higher average relative densities were measured with a corresponding increase in strength. Similarly, at a depth of 11.5 m, a higher relative density of 53% was measured with a corresponding increase in cyclic strength.

Figs. 6a through i also plot the cyclic strength of reconstituted specimens. For example, Fig. 6a plots the cyclic strength of reconstituted specimens prepared from sand at a depth of 2.5 m. While a number of tests were performed on sand from this depth, only test values for the two reconstituted specimens that had consolidated density values close to density values for the undisturbed specimens were plotted on the Figure. It may be seen that the cyclic strength of reconstituted specimens is generally lower than the strength of undisturbed specimens when test results are compared at about the same relative density.

Figs. 6b through i plot similar comparisons and it may be seen that at all depths, when relative density values are reasonably equivalent, the cyclic strength of undisturbed specimens is higher than the cyclic strength of reconstituted specimens.

Effect of Density on Cyclic Strength

To show the effect of relative density on cyclic triaxial strength, the data from Fig. 6 and from Table 1 for all depths and tested densities have been replotted in Fig. 7 which shows the stress ratio required to cause failure in different numbers of cycles (defined as ±5% double amplitude strain) versus the consolidated relative density of the specimen. Thus, Fig. 7a is a plot of the cyclic strength versus relative density of specimens that failed between 3 to 10 cycles.

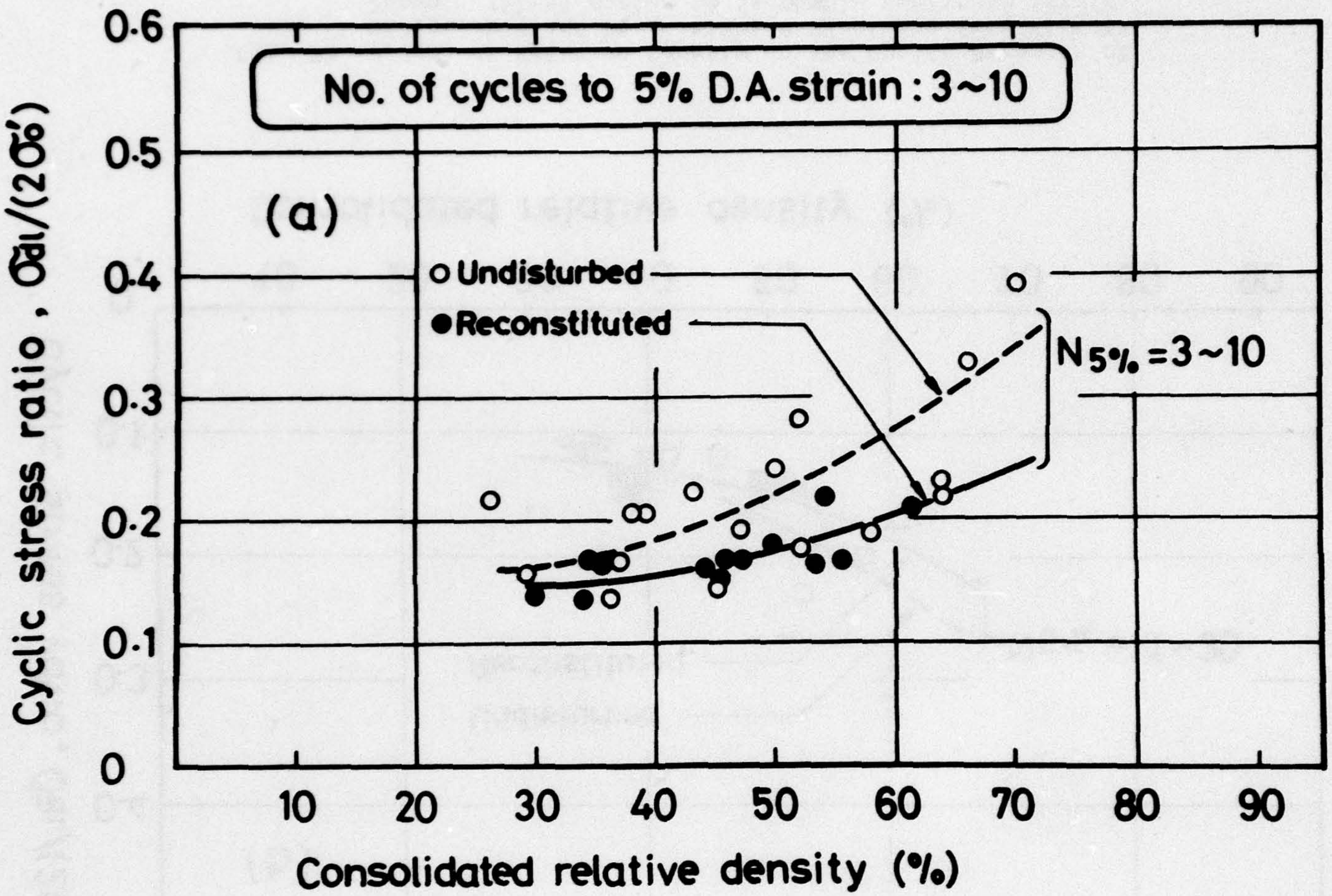


Fig. 7a Effect of Relative Density on the Cyclic Strength of Undisturbed and Reconstituted Specimens from Niigata, Japan. (3-10 cycles to 5% Double Amplitude Strain).

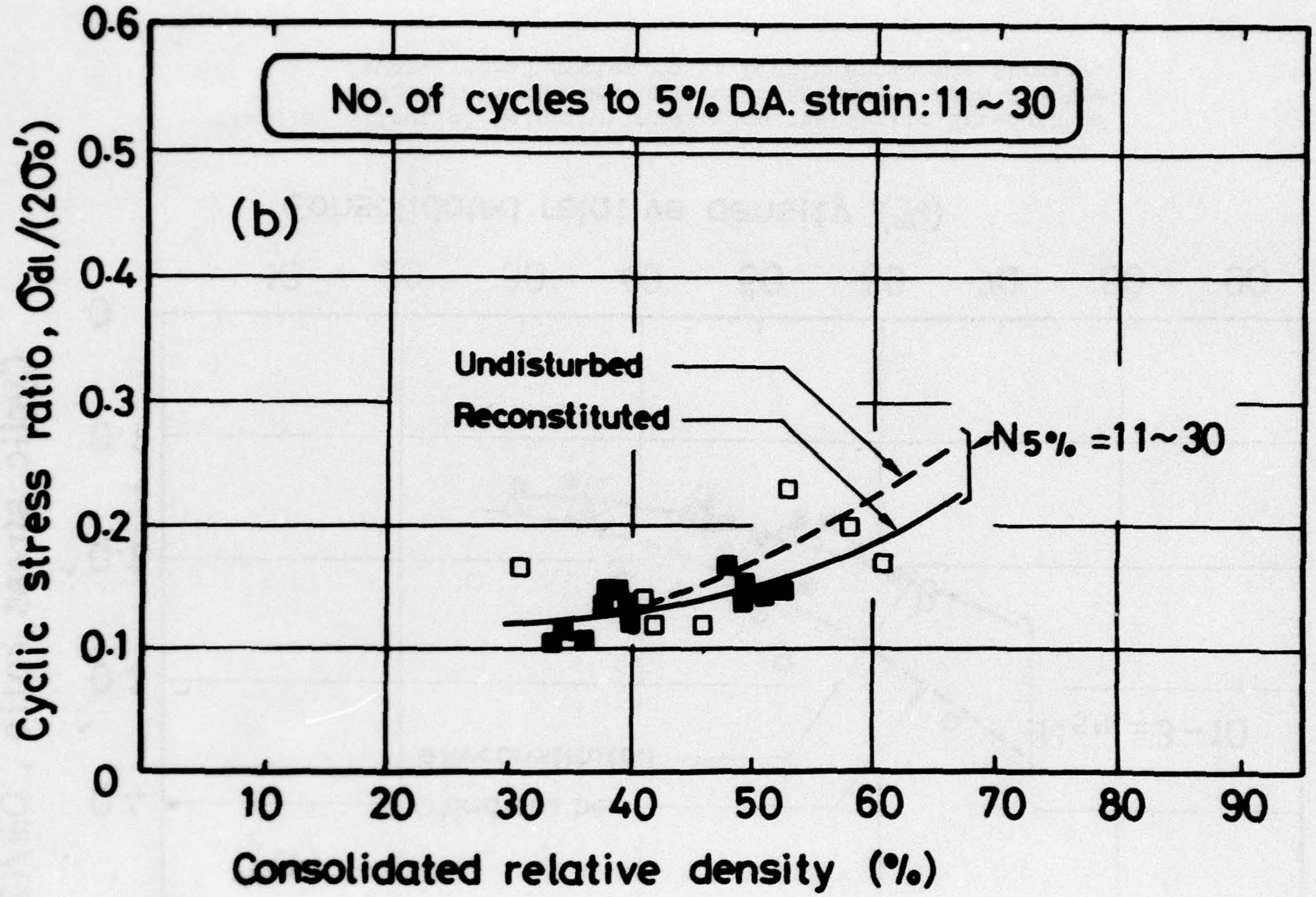


Fig. 7b Effect of Relative Density on the Cyclic Strength of Undisturbed and Reconstituted Specimens from Niigata, Japan. (11-30 cycles to 5% Double Amplitude Strain).

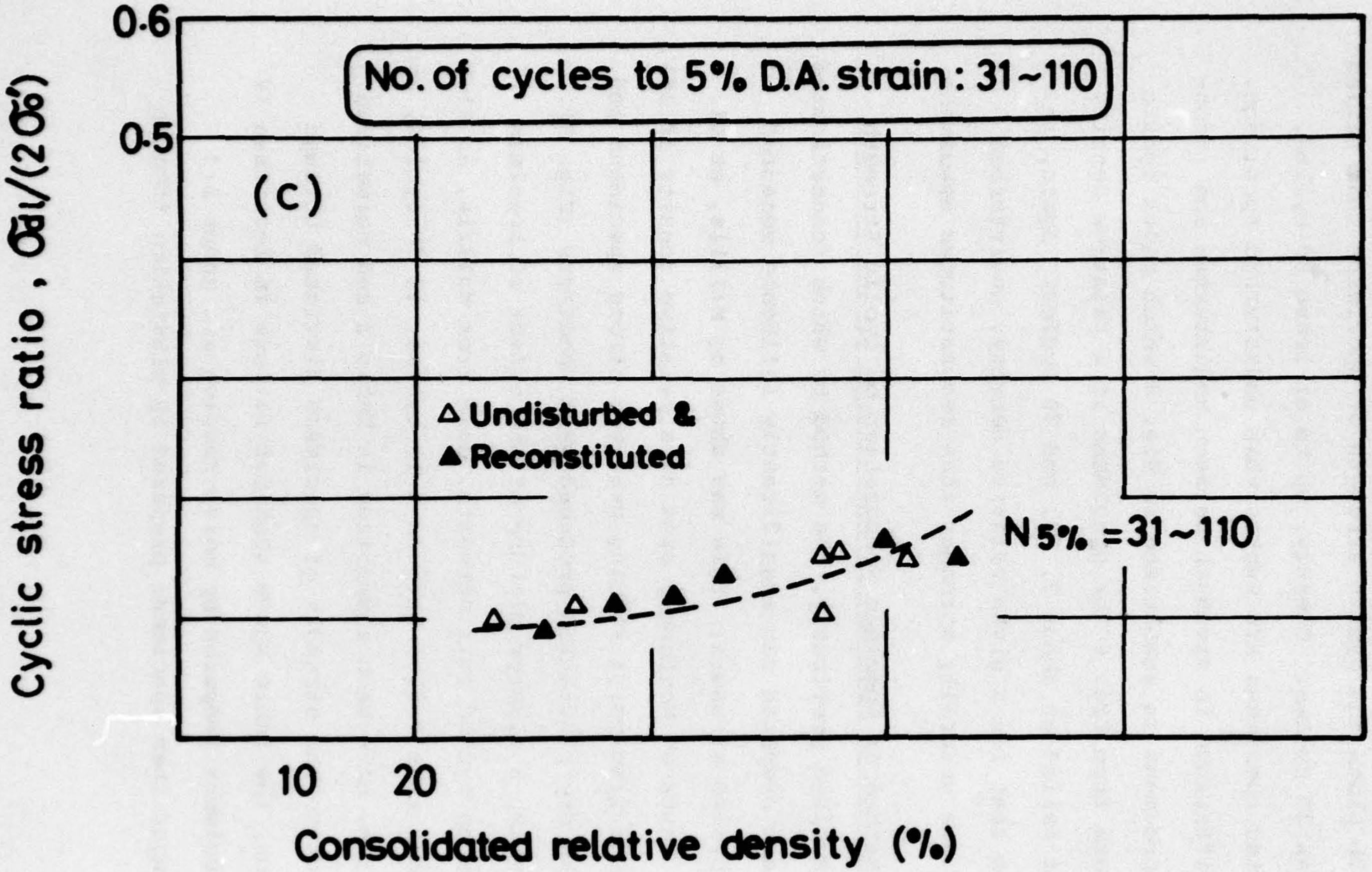


Fig. 7c Effect of Relative Density on the Cyclic Strength of Undisturbed and Reconstituted Specimens from Niigata, Japan. (31-110 cycles to 5% Double Amplitude Strain).

Fig. 7b plots the cyclic strength of specimens that failed in more than 30 cycles. Clearly, up to at least 30 cycles, reconstituted specimens are weaker than undisturbed specimens.

The difference in strength between undisturbed and reconstituted specimens is summarized in Fig. 8 which plots cyclic strength data from Fig. 6 for specimens at a relative density of 50% that failed at about 7, 20, and 70 cycles. Again, it may be seen that for a given relative density, undisturbed specimens were generally stronger than reconstituted specimens.

<u>Effect of Method of Specimen Preparation on Cyclic Strength</u>

As described previously, the method by which reconstituted specimens are prepared can significantly influence measured values of cyclic strength. This was shown by Mililis, et al. (1975) for tests on Monterrey sand at a relative density of 50%, where methods like moist rodding produced strong specimens and methods like dry pluviation produced weak specimens (Fig. 9).

To develop a clearer picture of the effect of specimen preparation on cyclic soil strength, data from Mulilis, et al, for specimens that failed between 3-10 cycles, 11-30 cycles, and 31-110 cycles, have been summarized in Table 2 and normalized with respect to the strength of specimens pluviated through water. Thus, the table shows that for failure in less than 10 cycles, specimens prepared by moist rodding are about 1.3 times stronger than specimens prepared by pluviation through

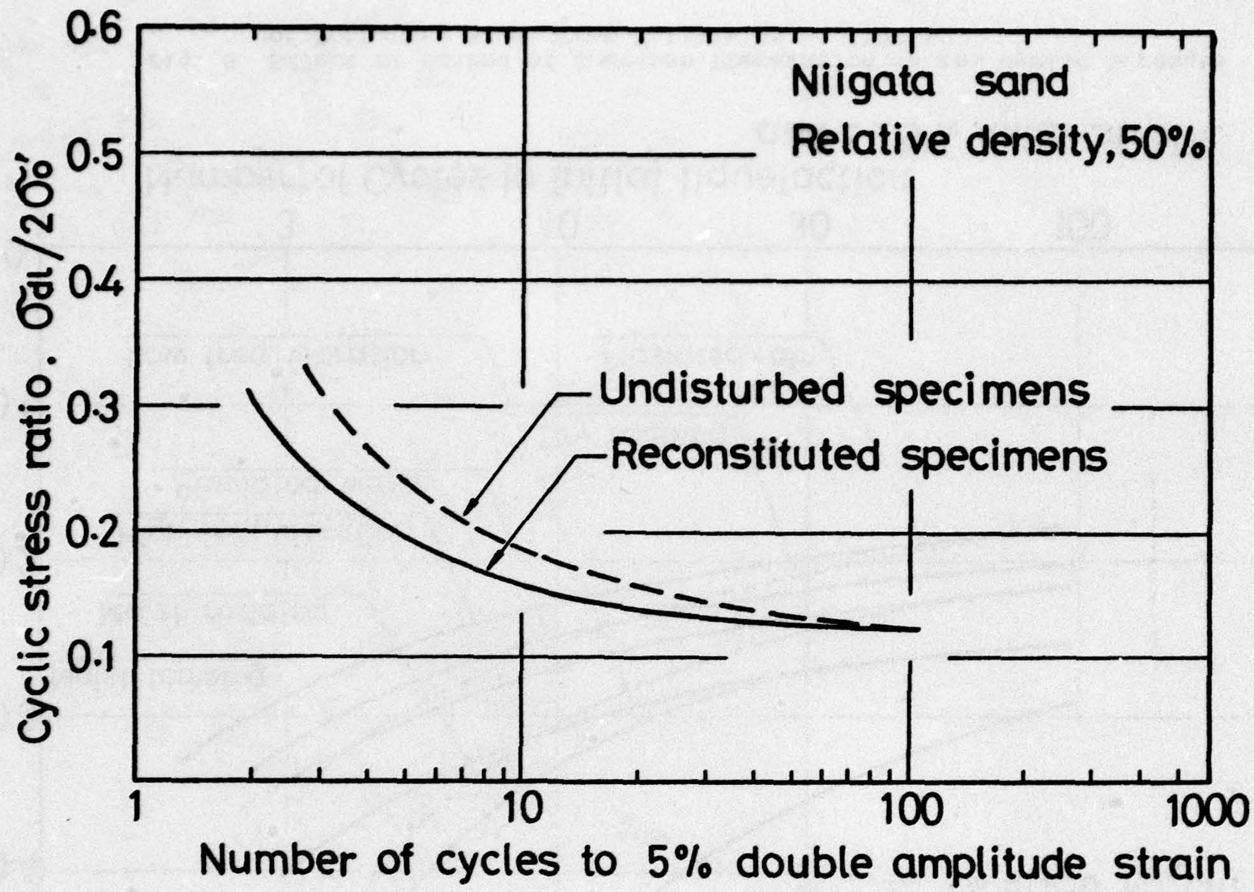


Fig. 8 Summary Curve Comparing the Cyclic Strength at 50% Relative Density of Reconstituted and Undisturbed Sand Specimens from Niigata, Japan.

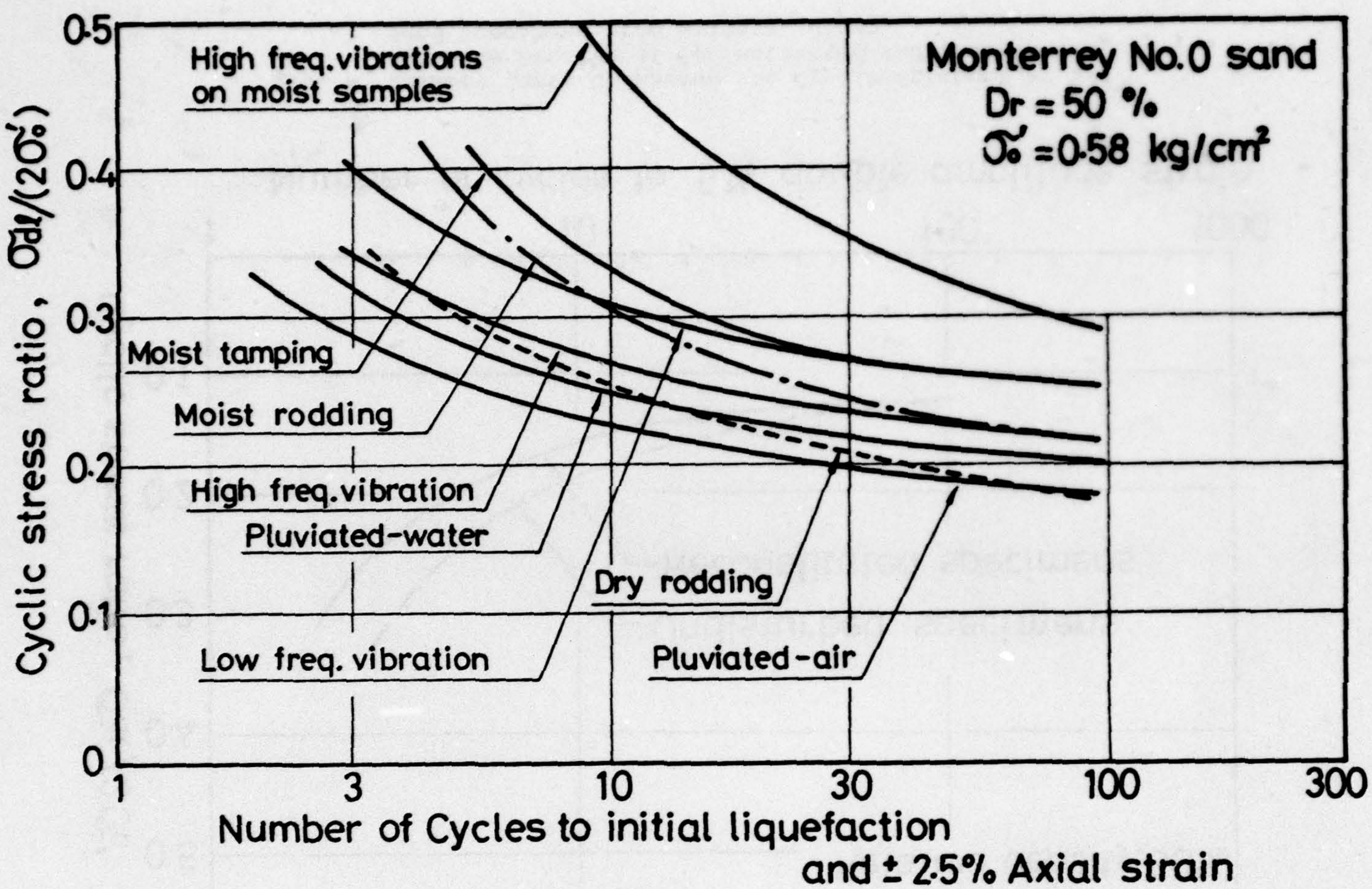


Fig. 9 Effect of Method of Specimen Preparation on the Cyclic Strength of Monterrey Sand (From Mulilis et al, 1975).

TABLE 2

SUMMARY OF THE EFFECT OF SPECIMEN PREPARATION METHOD ON THE CYCLIC STRENGTH OF MONTERREY NO. 0 SAND[1]

	Cyclic Stress Ratio at 2.5% Double Amplitude Strain			Cyclic Strength / Cyclic Strength for Specimens Pluviated in Water		
	N=3-10 5%	N=11-30 5%	N=31-110 5%	N=3-10 5%	N=11-30 5%	N=31-110 5%
Pluviated in Air	0.25	0.21	0.19	0.89	0.91	0.90
Pluviated in Water	0.28	0.23	0.21	1.00	1.00	1.00
Dry Rodding	0.30	0.23	0.19	1.07	1.00	0.90
High Frequency Vibration	0.30	0.25	0.23	1.07	1.09	1.10
Low Frequency Vibration	0.35	0.28	0.26	1.25	1.22	1.24
Moist Rodding	0.37	0.27	0.23	1.32	1.17	1.10
Moist Tamping	0.41	0.29	0.26	1.46	1.26	1.24
High Frequency Vibrations on Moist Samples	-	0.4	0.32	-	1.74	1.52

1 Adapted from Mulilis et al, 1975.

water. Other values may be read off the table to evaluate the effect of other numbers of cycles and other methods of specimen preparation on cyclic strength.

A summary similar to Table 2, but showing the cyclic strength of Niigata sand, can be plotted from Fig. 8. This Table 3 shows the cyclic strength of undisturbed specimens that failured in different numbers of cycles normalized with respect to the strength of reconstituted specimens prepared by pluviation through water. It may be seen that for failure between 3 and 10 cycles, undisturbed specimens are about 1.22 times stronger than reconstituted specimens. For failure between 11 and 30 cycles, undisturbed specimens are about 1.14 times stronger than reconstituted specimens and for large numbers of cycles (greater than about 50) the cyclic strength of undisturbed specimens is about equal to the strength of reconstituted specimens.

It is important to point out that the reconstituted specimens were prepared by pluviating sand through water. However, other specimen preparation techniques could have been selected for preparing stronger samples.

By comparing Table 2 and 3 it may be seen that the difference in strength between reconstituted specimens prepared by moist rodding and reconstituted specimens prepared by pluviating sand through water is about the same amount as the difference in strength between undisturbed specimens and reconstituted specimens prepared by pluviating through water. Thus, as a first approximation for Niigata sand, reconstituted specimens prepared by moist rodding techniques may better model insitu cyclic triaxial strength than specimens prepared by wet pluviation.

TABLE 3

Summary of the Difference in Cyclic
Strength Between Undisturbed Specimens and
Specimens Reconstituted by Pluviation Through Water

	Cyclic Stress Ratio		
	$N_{5\%} = 3 - 10$	$N_{5\%} = 11 - 30$	$N_{5\%} = 31 - 110$
Undisturbed Specimens	0.18	0.14	0.12
Reconstituted Specimens (pluviated through water)	0.22	0.16	0.12
Undisturbed strength / Reconstituted strength	1.22	1.14	1.00

Application of Results to Other Sands

To help determine how the tests results reported on the previous pages can be applied to other sands, cyclic triaxial strength tests were performed on Monterrey No. 0 sand. These specimens were prepared by pluviation through water in the same way as for Niigata sand. These test results are shown in Fig. 10 where it may be seen that Monterrey No. 0 sand is slightly stronger than the Niigata sand. Nevertheless, the shapes of the two curves agree well together.

Therefore it would seem reasonable to assume that insitu cyclic soil strength is better modeled by testing reconstituted specimens prepared by wet rodding or similar specimen preparation techniques that give higher cyclic strengths than by testing reconstituted specimens prepared by wet pluviation that give lower cyclic strengths.

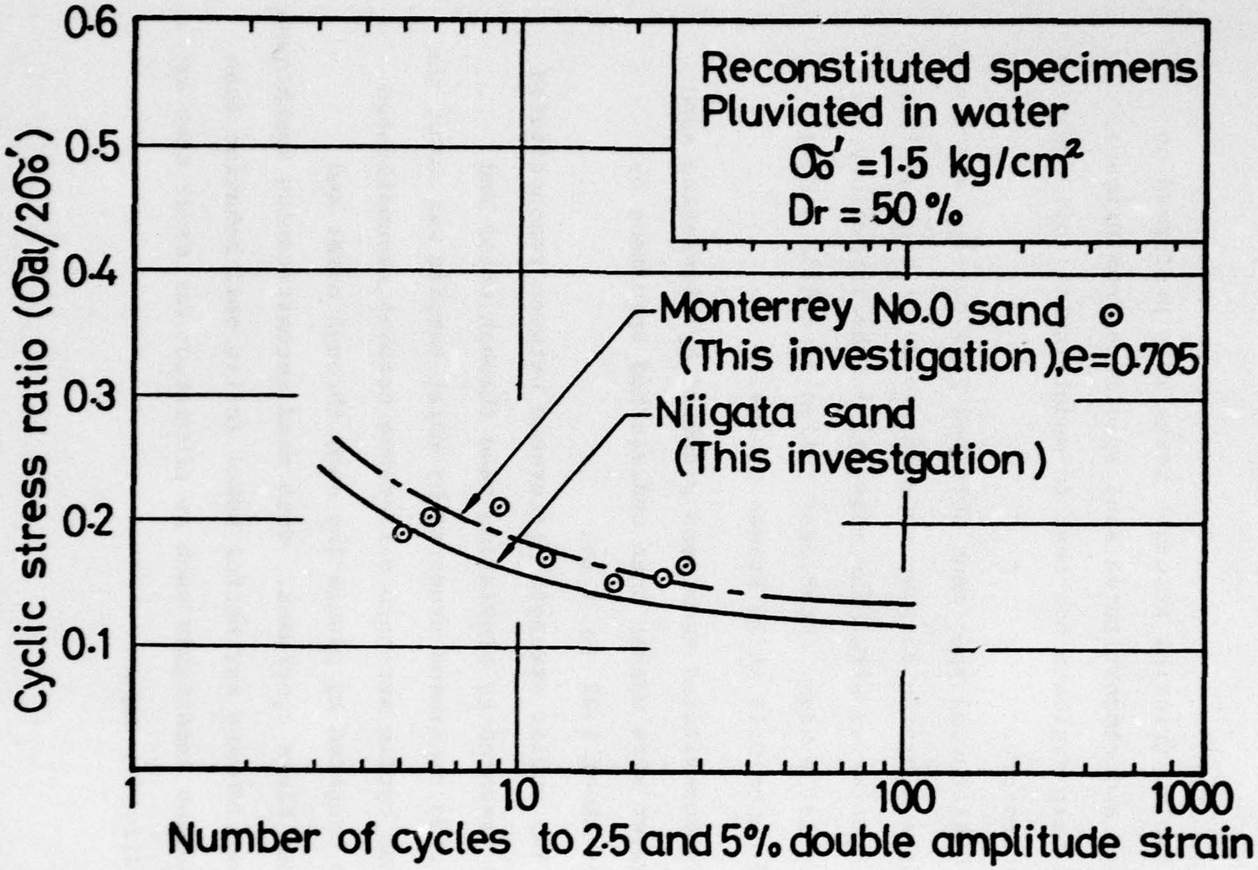


Fig. 10 Comparison of the Cyclic Triaxial Strength of Reconstituted Monterrey Sand and Niigata Sand Prepared by Pluviation in Water.

CHAPTER 4

SUMMARY AND CONCLUSIONS

1. Cyclic triaxial strength tests were preformed on undisturbed and reconstituted sand specimens from Niigata, Japan, to help evaluate how best to model insitu soil behavior in the laboratory.

2. Undisturbed specimens obtained from careful sampling with a large diameter sampler appeared to be of high quality, yet cyclic triaxial strengths measured in the laboratory were not particularly high. Specimens failed at cyclic stress ratios of about 0.15 at 20 stress cycles.

3. Reconstituted specimens prepared by pluviating sand through water were weaker than undisturbed specimens by factors of about 1.22 to 1.16.

4. The cyclic strength difference between reconstituted specimens prepared by pluviating sand through water and reconstituted specimens prepared by moist tamping was about the same as the cyclic strength difference between reconstituted specimens prepared by pluviating sand through water and undisturbed field specimens. Thus sand reconstitution techniques such as wet tamping may better model insitu soil behavior than reconstitution techniques such as pluviation for sands such as those at Niigata, Japan.

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Prepared for Office, Chief of Engineers, U. S. Army, Washington, D. C., under Contract No. DACW39-76-M-2407.
References: p. 47.

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